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INCLUDING PRESTRESSED CONCRETE

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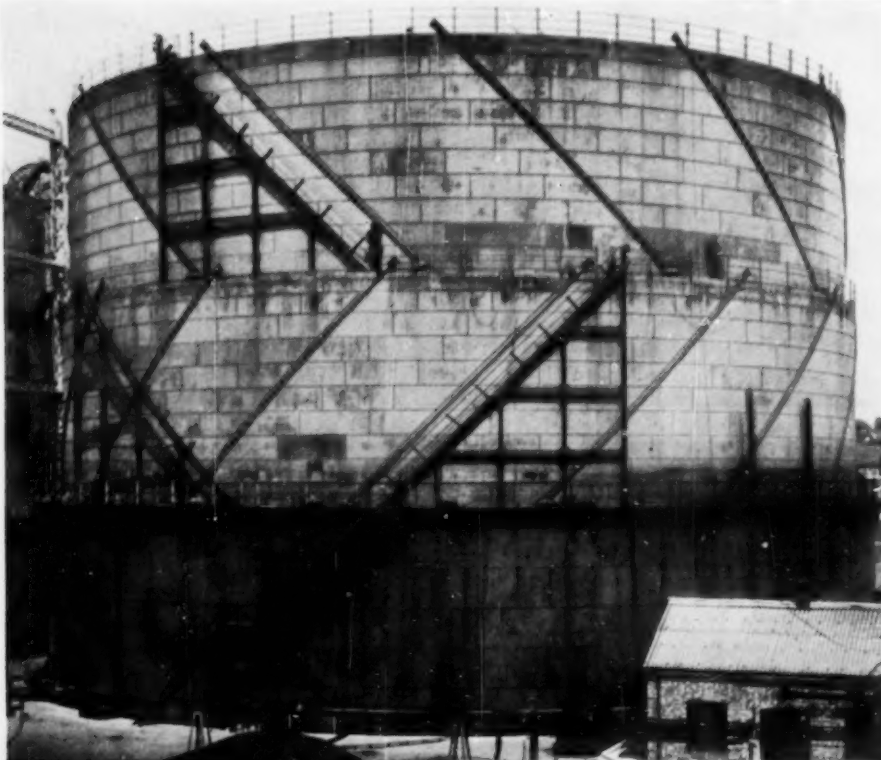

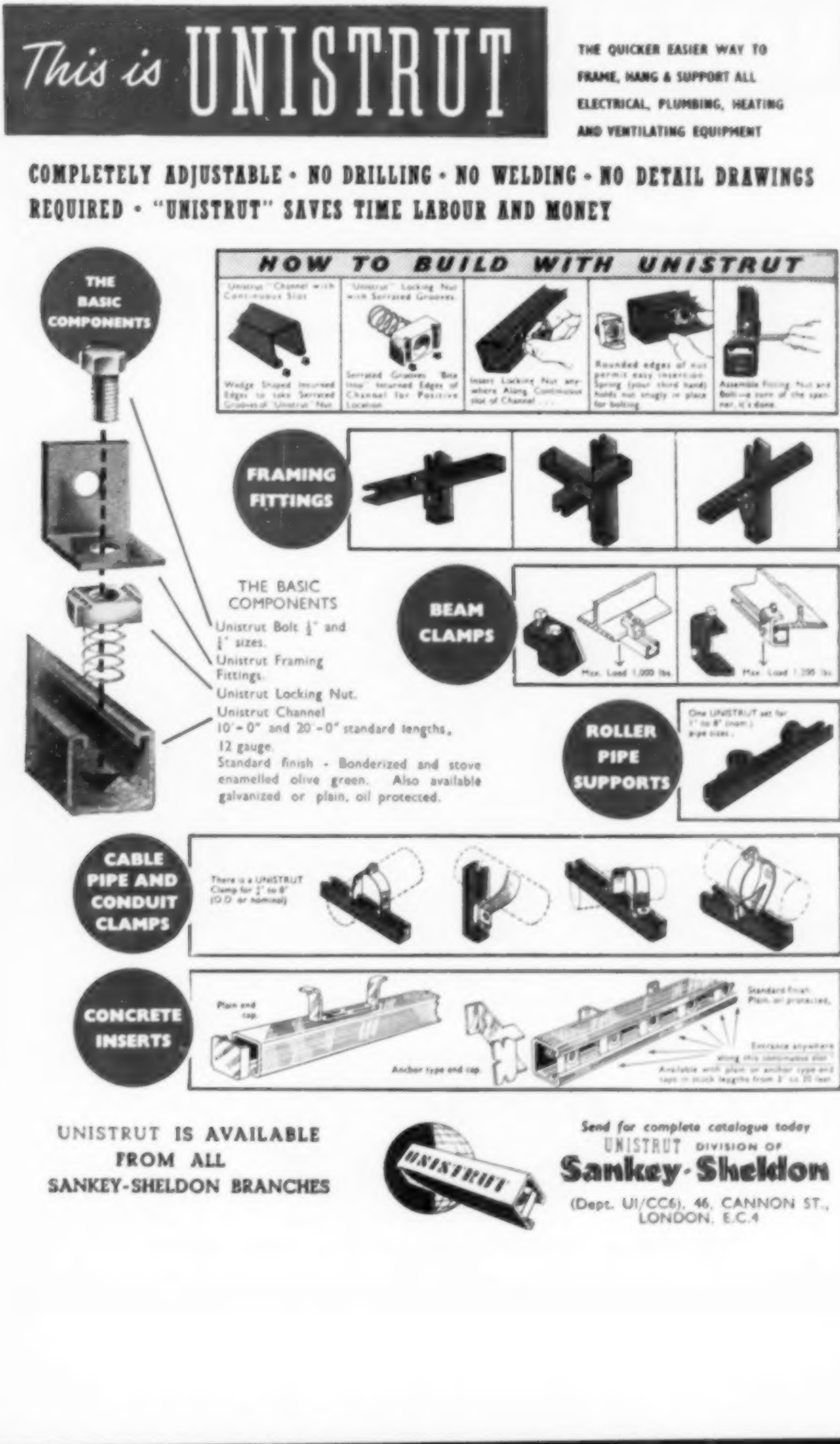
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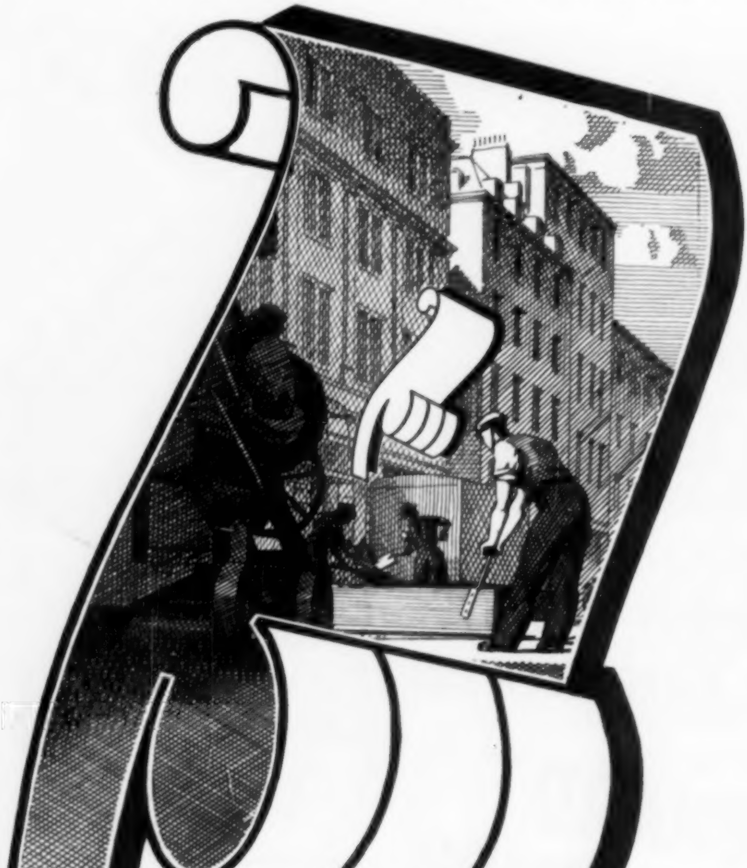

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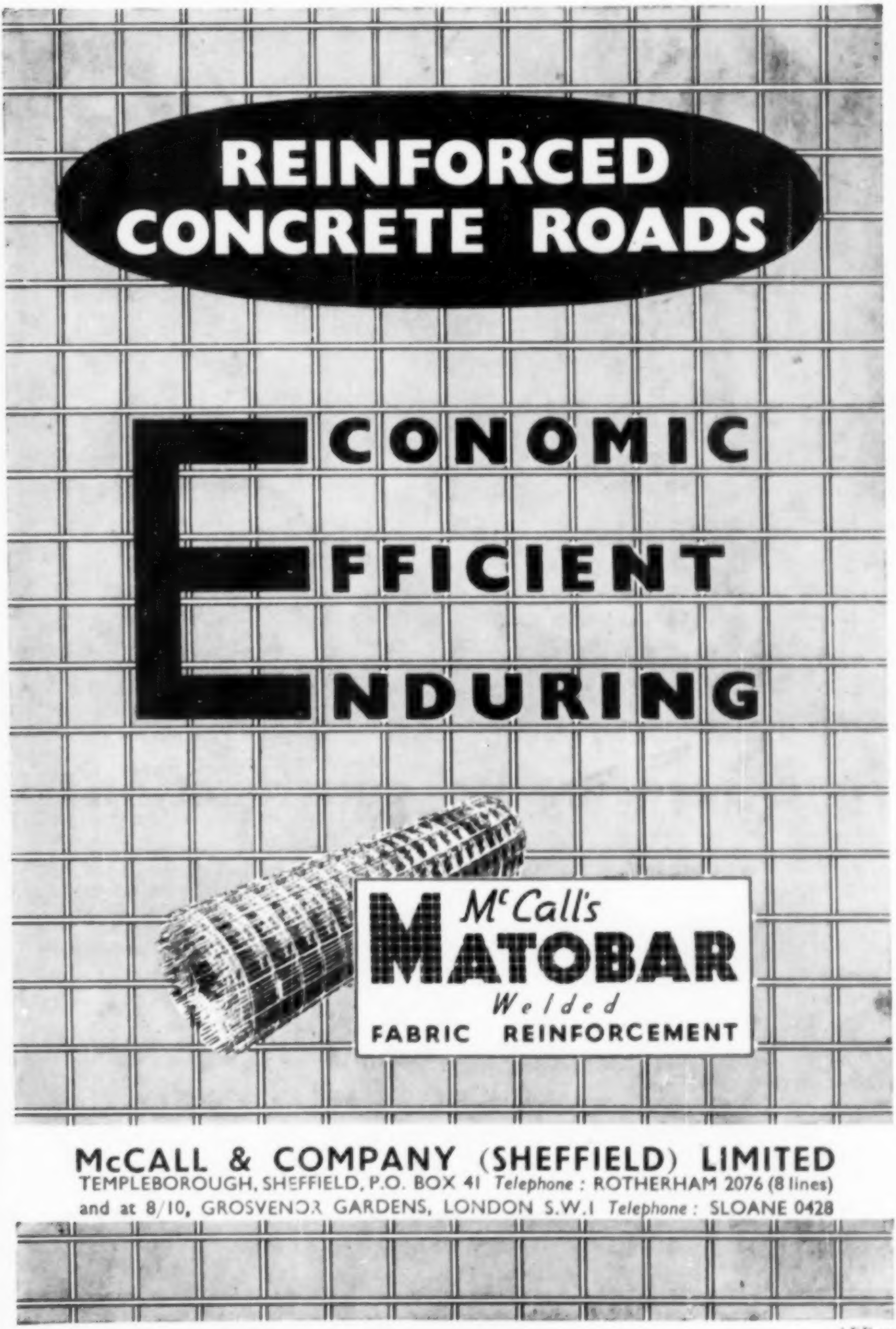
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# CONCRETE
## AND
## CONSTRUCTIONAL ENGINEERING

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Volume L, No. 1. LONDON, JANUARY, 1955.

# *EDITORIAL NOTES*

## Design, Decoration, and Utility.

SINCE the introduction of reinforced concrete made possible the use of concrete in all classes of structures, and in all architectural styles old and new, the development of this material has been hindered because many architects consider it to be a utilitarian material only. Many papers have been read and much has been written by architects in praise of concrete as a material with unlimited possibilities for new shapes, decoration, and architectural features moulded economically in the shuttering or precast, but in Great Britain little has been done in this direction compared with the "architectural concrete" structures that are a prominent feature among the illustrations in the architectural press of the U.S.A. In America and elsewhere reinforced concrete is used freely for monumental buildings of all kinds, and particularly in their adornment with decorative features and sculpture either moulded in the shuttering or precast and fixed to the façades. In this country, on the other hand, concrete is most commonly used for plain walls only.

It was shown in this journal for November, 1954, that the Council of Industrial Design also takes the view that cheapness and utility are the primary requisites of concrete lamp posts. On page 70 of this number is given a letter from a member of the Street Furniture Committee of the Council, which was responsible for the designs, and a letter from the secretary of the committee that was published in another journal in which these new designs were criticised. These letters give an interesting insight into the confusion of thought on which these designs are based. In one sentence the secretary states that the committee is not against decoration, and in another he says that the committee favours elegance of proportion and line rather than additional ornament. The member of the committee says that the committee is not opposed to decoration where it is appropriate, and in the official journal of the Council it is stated that no effort has been spared to produce posts devoid of unnecessary ornament. This can only mean that while the committee does not disapprove of decoration it will not have it, and the result is seen in the plain posts that have been approved and of which photographs have been issued by the Council; in every case the posts are as plain as a gas barrel. An approach to art that sees beauty in functionalism and line only, and a desire for the utmost cheapness, are the ruling factors in the design of these posts. The possibility of adding a little pleasing decoration at negligible extra cost has been ignored. There seems to be an idea

that the bare requirements of efficiency must necessarily be beautiful, that beauty is derived from geometry rather than from nature, and that a straight-edge and a set of French curves are the only tools required by an artist. This is the exact opposite of the approach of the great artists of the past and of the present who derive their inspiration from nature, which is said to abhor a straight line. No one will deny that its ability to serve its purpose in lighting a thoroughfare is the first essential of a lamp post, but we see no reason why only the plainest and cheapest of posts should be erected all over the country. In such conspicuous objects as tall lamp posts more is necessary than the dull uniformity of a standardised product. The changes in styles of building as well as of scenery are among the pleasures of travelling. Standardisation is monotony, and monotony is dull. The desire for economy at the expense of a little pleasure is out of place so long after the war when the saving of labour and materials was of paramount importance. For a good many years now local influences have been disappearing as more and more houses, schools, and hospitals are built all over the country in accordance with manuals and instructions prepared and issued by Government departments in London. This is a pity.

The Council of Industrial Design claims that the restriction of choice of the design of lamp posts is a democratic procedure. The word democratic has, however, different meanings to different people, and freedom to choose only from approved designs is very like an election in which one can vote only for a nominee of an existing government. The membership of the Street Furniture Committee comprises a representative of the Road Research Laboratory, a city engineer, an editor of an architectural journal and the secretary of the Modular Society (both of whom are also members of the Royal Institute of British Architects), a practising architect, and the director of the Council of Industrial Design. This seems a very unrepresentative committee empowered to force its will upon the community under pain of sanctions in the withholding of grants by the Government. Decoration that this committee condemns as a waste of money may to many people be money well spent. The dull monotony that is an inevitable result of standardisation in buildings and street furniture is quite unnecessary when decoration can be so cheaply applied to moulded concrete products made in large numbers in steel moulds. Since Corbusier declared that a home should be merely a machine to live in there has been a tendency to consider functionalism as the only requisite of art. Utility and line are not the only criteria of objects that one cannot avoid seeing whenever one goes out of doors. We agree with our correspondent when he says that "inept" decoration is undesirable, but why must it be inept? Have we no artists to-day who could, in collaboration with the designers of the moulds, produce pleasing decoration that would not be expensive? We believe that there are plenty of such artists, but it seems that they are not employed because of the decision of the Council that these posts must be "devoid of unnecessary ornament."

Standardisation is useful in many directions, but it should have no place in art. In matters of taste people should be allowed to have what pleases them rather than what a committee or a Civil servant thinks is good for them. It would be interesting to see a design for a lamp post by the President of the Royal Academy, whose views on this subject are given on page 20.

# "Folded-Plate" Roofs in the U.S.A.

By MILO S. KETCHUM.

THE roofs of a reinforced concrete building (*Figs.* 1 and 2) constructed in Denver, U.S.A., for the H. W. Moore Equipment Company consist of Z-shaped and inverted V-shaped "folded-plates". [The term "folded-plate" is used to describe this type of roof in preference to "hip roof" or "prismatic roof" as in the writer's opinion it more nearly conforms to the original German description of "Faltwerke".]

The building comprises an office and storage area, a workshop for the repair of construction equipment, and a display and utility area. In the workshop area heavy crane gantries were required which would have had to span 80 ft. unless they were supported by the roof as well as by columns at the ends of the gantries.

**Fig. 1.—Building with "Folded-plate" Roof.**

**Fig. 2.—The High Crane Bay and Offices.**

A reinforced concrete folded-plate roof was therefore selected because it permits concentrated loads to be suspended from the roof. In the office and display area a mezzanine floor is suspended from the roof.

The roof over the workshop (*Fig.* 3) is formed by inverted V-shapes of 80 ft. span and 36 ft. wide between the valleys. The height of the valleys above the floor is 20 ft. except in a bay where an elevated crane is installed and where the height is 26 ft. The thickness of the roof is 6 in. except at the supports where it is increased to 8 in. to resist shearing forces. Doors comprise a great part of the elevations so that work may be done outside in fine weather, which exists most of the time in Colorado.

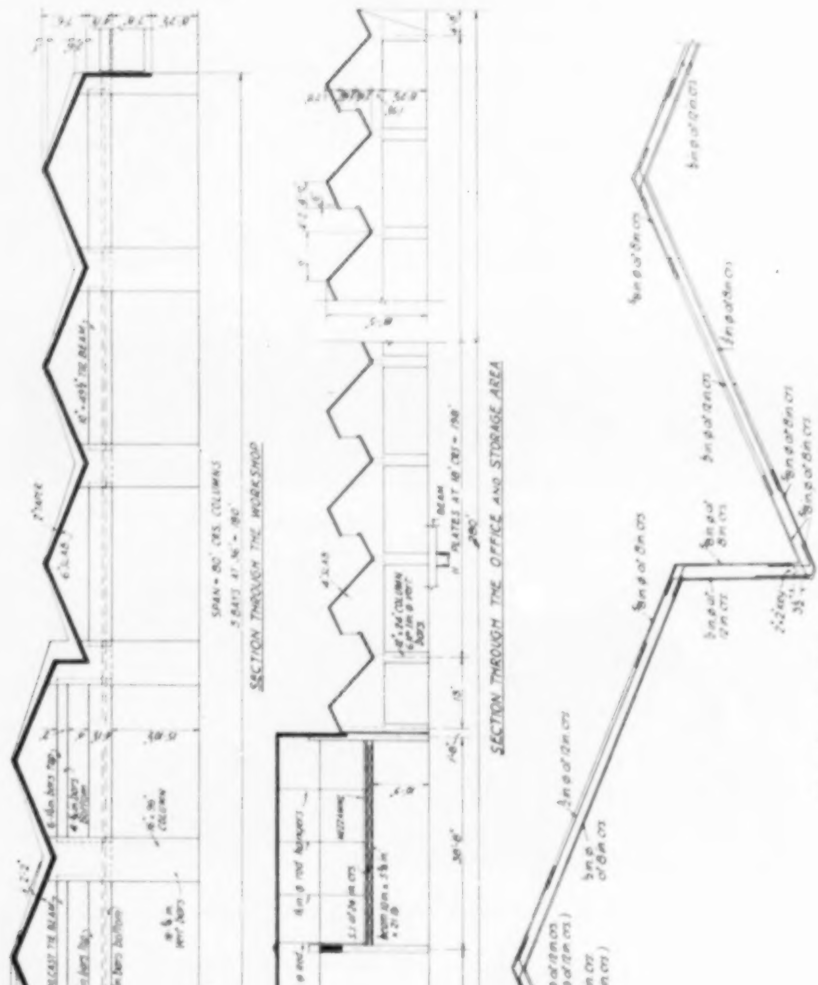



**Fig. 3** *(top)*, **Fig. 4** *(centre)*, **Fig. 5** *(bottom)*.

The office and storage area is covered by a Z-shaped roof which has a span of 75 ft. with a 22-ft. cantilever at each end (*Figs.* 4 and 6). This roof is arranged so that north-light clerestories are provided. The slabs are 4 in. thick and are stiffened by reinforced concrete end-walls and columns. The roof of the display area (*Fig.* 5) is similar to that of the workshop area except that the span between the valleys is 40 ft.

The forces in the roofs were analysed by the method described by Winter and Pei * but with the modifications for deflection of plates suggested by Gaafar,† which is analogous to the corrections for side-sway in rigid frames. These modifications, in the case of the workshop, affected appreciably only the bending moments in a transverse direction in the roof over the bay with the elevated crane ; in other roofs of the workshop the deflection of the slabs is balanced so that there is no differential movement at the ends. At the junction of the higher and lower roofs the modification was greatest, some bending moments being

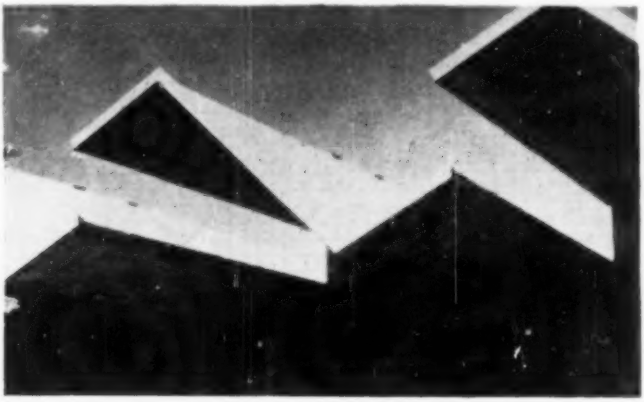


**Fig. 6.—The Cantilevers over the Office and Store Building.**

completely reversed. However, when the areas of steel were checked it was necessary to change very few sizes or spacings of the bars, because both the top and the bottom of the slab were designed with a minimum percentage of steel. Some details of the reinforcement are shown in *Figs.* 3, 7, and 8. The Z-shaped roof was not affected by these modifications as the bending moments are statically determinate.

Inclined stiffeners at the supports of the inverted V-shaped roof act as struts to transfer load to the ties near the lower part of the roof. Wide columns were used in the workshop area (*Fig.* 3) for housing the sliding doors. The width need not have been so great if strength had been the only consideration ; nevertheless, the width enabled the size of the stiffeners to be reduced because of the resistance of the columns to eccentric thrusts from the roof. Larger ties are required at the ends of the workshop area than at the intermediate spans because the thrusts due to dead loads are balanced in the intermediate spans. Loads

* George Winter and Minglung Pei, "Hipped Plate Construction". A.C.I. Journal, January, 1947, pages 505–32.
† Ibrahim Gaafar, "Hipped Plate Analysis Considering Joint Displacements". Proceedings A.S.C.E., Vol. 79, Separate No. 182, April, 1953.

from the crane rails are transferred to the roof close to the transverse joints so that the bending moment on the slabs is only slightly affected.

Bored concrete piles were used to form the foundation for the office and storage area, and timber piles for the workshop area. The bearing stratum is about 25 ft. below the floor level. The outer walls are supported on beams spanning between the piles.

Competitive tenders were invited for the construction of the building. Because of the unusual nature of the construction, complete designs for movable centering were made and incorporated in the tender documents, to be used at the option of the contractor. The centering consisted of steel trusses, carrying wooden joists, supported at four places and jacked into position. Removal and re-fixing of the centering were done by moving them sideways rather than moving them out

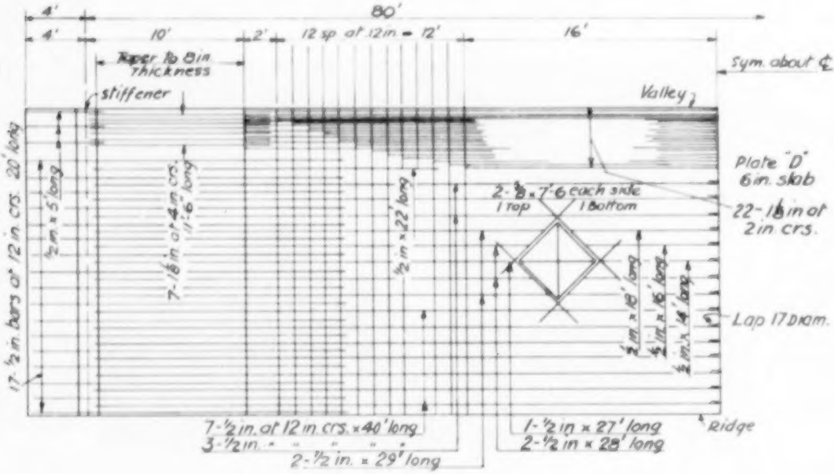



**Fig. 7.—Details of Reinforcement.**

**Fig. 8.—Reinforcement for a V-shaped Roof.**

of the building and back again. At the centre of each span a timber shutter was provided which could be left in place to serve as a prop when the centering had been removed. The contractor used the centering designed by the engineer except for one end of the office and storage area roof and some parts of the V-shaped roof where wooden shutters of ordinary design were used. Some of the trusses were re-designed so as to be of constant depth and have a greater salvage value, or so that they could be used for constructing similar roofs of different spans.

The compressive strength of the concrete was assumed in designing the roofs to be 3750 lb. per square inch, and a maximum slump of about 2 in. was required for the steepest parts of the roofs. The strength of the concrete as used was over 5000 lb. per square inch (tested on cylinders) at 28 days. There was no risk of the mixture being too wet because it would not stay on the slope if it contained more than the specified water content.

The architect was Mr. Tom Moore of Denver, the structural engineers were Messrs. Ketchum & Konkel, of Denver, and the contractors the N. G. Petry Construction Company.

# Book Review.

**"Design of Concrete Structures"** by L. C. Urquhart, C. E. O'Rourke, and G. Winter. (London: McGraw-Hill Book Co. 1954. Price £3.)

It is surprising that text-books dealing with reinforced concrete and published in the U.S.A. do not, in general, mention the application of reinforced concrete to industrial structures such as bunkers, gantries, silos, reservoirs, and elevated tanks. The fifth edition of this book, although a useful introduction to the design of reinforced concrete as applied to buildings and bridges, is no exception, and one is left with the impression that in the U.S.A. reinforced concrete is confined to the construction of buildings. Possibly the explanation is that books of this nature are written to provide texts for courses in colleges and universities rather than to be used as works of reference by engineers in practice.

Methods of calculating stresses in slabs, beams, and columns under axial loads are clearly described, but the analysis of columns subjected to bending and compression is confined to the algebraic or graphical solution of the cubic equation for *n*. The more generalised iterative solutions such as those due to Wessman and Bakhoum, the application of which are not confined to rectangular or circular members, are not mentioned, nor are the possible advantages of using a different amount of reinforcement in the face of a column subjected to tensile stresses compared with that used in the compressive face. Detailed examples of the design of a beam-and-slab floor and a flat-slab floor are given. Foundations and retaining walls are well treated; the only minor criticism is that it is implied that the code for the use of reinforced concrete published by the American Concrete Institute does not require any bars in column bases to be hooked, whereas in fact the A.C.I. code does require plain round bars to be hooked.

The chapter on highway bridges is short but excellent, and there are notes on composite construction of structural steel and reinforced concrete slabs. It is surprising that this latter method of construction has largely been used only for bridges as there are possibilities of considerable savings in steel were it to be applied to buildings. The last chapter deals with developments in design and provides a clear explanation of the methods due to Jensen and Whitney of calculating the ultimate strength of a member subjected to bending. Prestressed concrete is dealt with briefly but adequately, and it is pleasing to see that the authors manage to do this without the use of esoteric symbols. The book generally is well written, although exception might be taken to the misuse of the word "insulation" instead of the shorter and more generally used word "cover", and the occasional use of the word "medium" for "material".—J. E. G.

# A School Assembly Hall.

The Tettenhall comprehensive school, near Wolverhampton, will accommodate 825 pupils and the buildings will cover an area of 71,417 sq. ft. Shell roofs are to be used for the workshops and the main assembly hall. *Fig.* 1 shows sections through the assembly hall which will have a roof 3 in. thick curving in two directions and also sloping from the balcony towards the stage. The hall is 104 ft. long by 52 ft. wide. The radius of curvature is 50 ft. across the width and 360 ft. longitudinally. The edge-beams are on columns at 13 ft. centres; there are also intermediate columns under the stiffening beams. The underside of the roof will be coated with sprayed vermiculite ⅝ in. thick; two layers of roofing felt will form the external finish.

The column bases are to be founded on sandstone which occurs over the site at a fairly uniform depth. The bearing strength is assumed to be 4 tons per square foot. All exposed columns will have a circular core of 16 in. diameter and will be enclosed in a 12-sided fibrous-plaster casing. Rainwater pipes are to be within the columns. The balcony will be supported on raking beams which will rest on the upstanding stiffening beams of a lower shell spanning transversely between the side columns. The concrete mixture specified for all the reinforced concrete is 1 : 2 : 4.

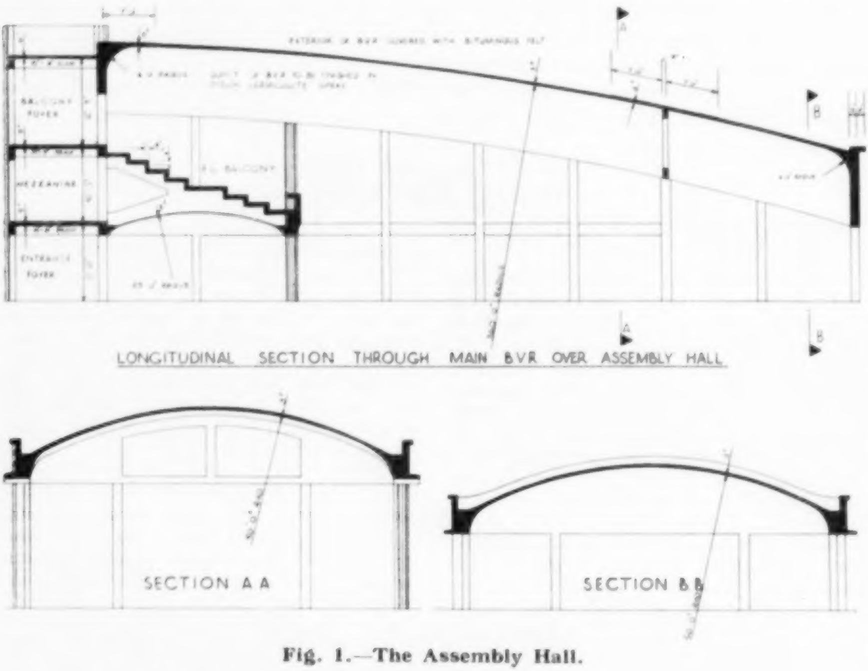



**Fig. 1.—The Assembly Hall.**

The work is under the direction of Mr. A. C. H. Stillman, F.R.I.B.A., Architect of the Staffordshire Education Committee. The contractors are Messrs. McKeand, Smith & Co., Ltd. The reinforced concrete is being designed by B.V.R. (Designs), Ltd., and the reinforcement, which consists of high-tensile square twisted bars and high-tensile fabric, is being supplied by "Twisteel" Reinforcement, Ltd.

# A Large Factory at Welwyn.

## SHELL ROOFS BUILT WITH TRAVELLING CENTRES.

THE first part, covering 100,000 sq. ft., of a factory for Murphy Radio at Welwyn Garden City is now nearing completion, and work was recently commenced on an additional 40,000 sq. ft. This is part of a factory which will eventually have a floor area of about 300,000 sq. ft.

The production shop (*Figs.* 1 and 2) is 470 ft. long by 180 ft. wide. The walls comprise reinforced concrete columns and beams with brick cladding. The roofs are of cantilever shell construction generally 2½ in. thick; there are clerestories above the shells, and the glazing faces east and west.

Sixteen double cantilevers (*Fig.* 3), each 180 ft. long, cover the building, and are supported by stiffening beams at 60-ft. intervals with prestressed concrete ties at 10 ft. centres between the ribs at the bottom of the clerestories. The internal columns are 2 ft. by 1 ft. in cross section at floor level and increase in width to 3 ft. 6 in. at roof level. The supports at the ends of each bay are formed by two columns 1 ft. square. Nine travelling shutters (*Fig.* 4), each 20 ft. long, were used for constructing the roofs. This comprised a main carriage and two movable wings. The carriage travelled on nine wheels, and fine adjustments to the wings were made by hydraulic jacks and steel wedges. The wings were supported at the inner side by hinges and at the outer edge by twin tubular-steel struts which terminated close to the top of the double-channel stanchions of the main carriage. When the wings were lowered the shutters were moved forward by winches to the next position. The roof of the production building was erected in thirty-four weeks. Each of the travelling shutters weighed about 8 tons. The shutters were then turned through 90 deg., for the construction of the roof of a second building.

The roofs are insulated against loss of heat by a layer of cork and double glazing. Insulation against excessive rise in temperature inside the building is provided by the white mineralised felt surface of

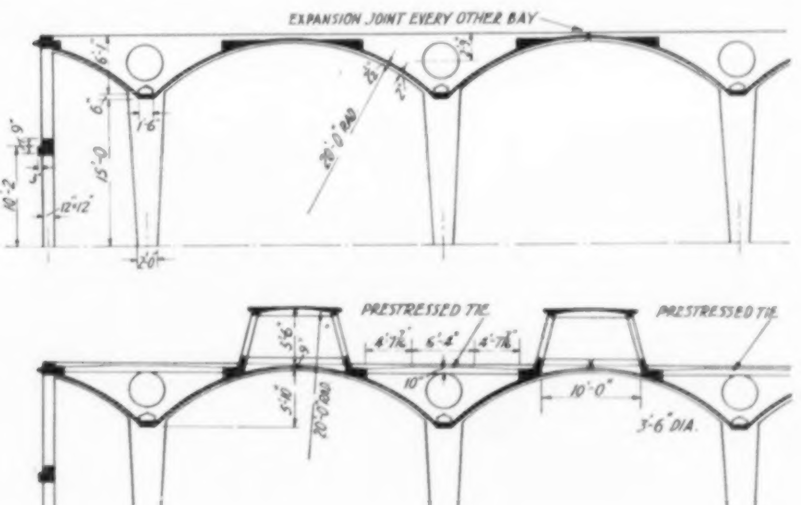



**Fig. 1.—Cross Sections.**

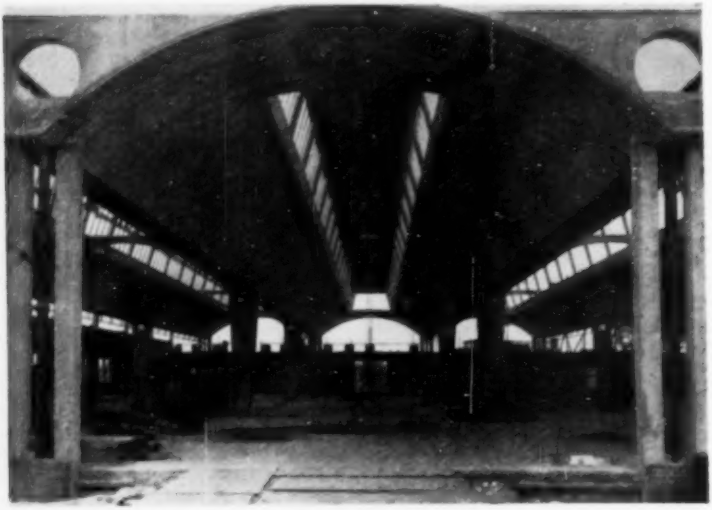


Fig. 2.—A Bay in the Production Building.

Fig. 3.—Concreting a 60-ft. Section of the Roof.

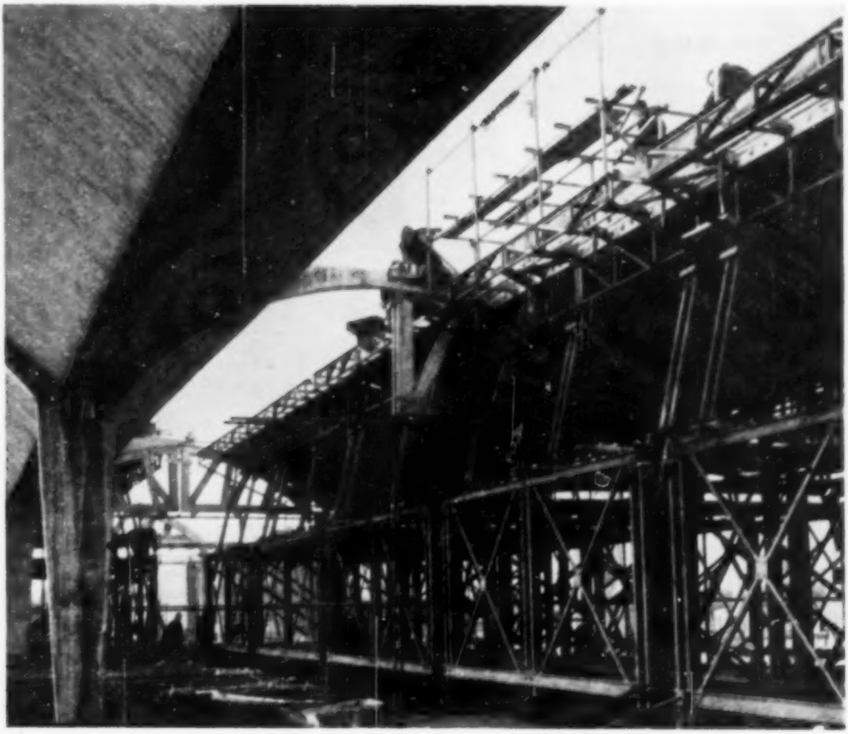


Fig. 4.—The Travelling Centering.

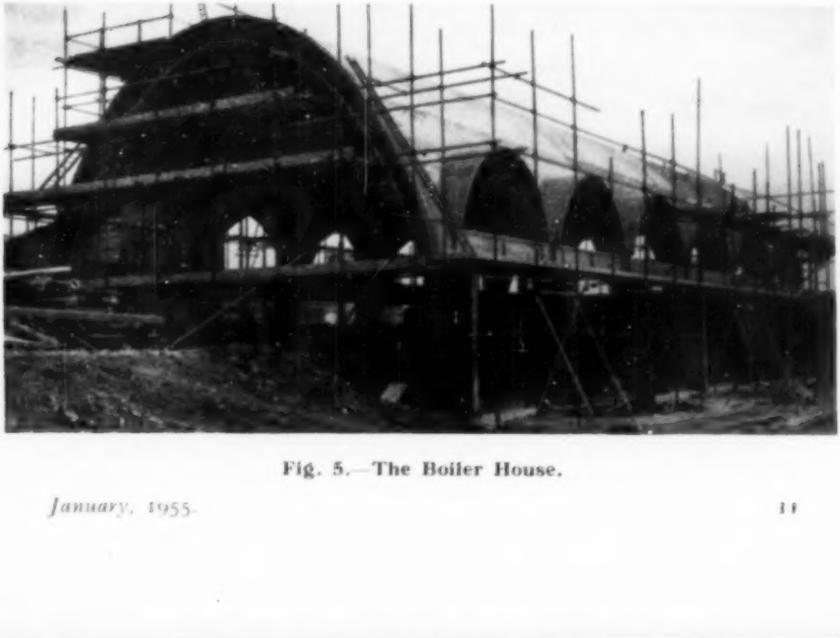


Fig. 5.—The Boiler House.

the waterproof covering. The building will be artificially ventilated with filtered warm air.

The factory is on a gently-sloping site. Part of the reinforced concrete floor is being constructed on hardcore, and a suspended floor is constructed over a large area.

The boiler-house (*Figs.* 5 and 6) is about 90 ft. long, 40 ft. wide and 30 ft. high and houses four boilers for high-pressure hot water. Hot water is taken to the factory by overhead pipes, and this allowed the pumps to be placed at first-floor level with the main water-storage tanks immediately overhead; the air compressors, transformer room, and switch-rooms are below the pump room. The building has an elliptical roof with dormer windows on each side, the ends having patent glazing. Two oil-storage tanks each 30 ft. long by 9 ft. diameter will be installed below ground adjoining the boiler-house.

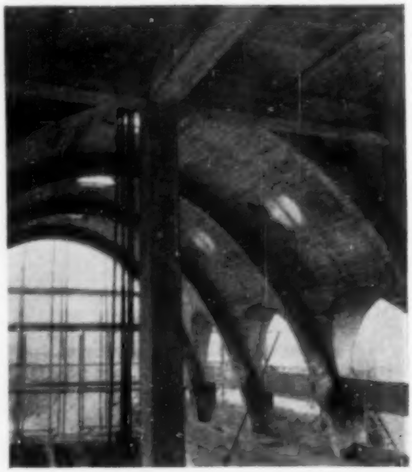


**Fig. 6.—Interior of Boiler House.**

The architect is Mr. C. W. Hutton, B.Arch., F.R.I.B.A. Mr. H. G. Cousins, B.Sc., M.I.C.E., M.I.Struct.E., is the consulting engineer for the structural work and Messrs. Handcock & Dykes for the heating, ventilating, and electrical installations. The general contractors are Messrs. John Laing & Son, Ltd.

## Tampa Bay Bridge, U.S.A.

THE bridge across Tampa Bay, Florida, has now been completed. It is three miles long, and is built on the Lee-McCall prestressing system in accordance with a design submitted by Mr. Donovan H. Lee, M.I.C.E., and accepted in competition with designs using tensioned stranded wire and 0·2-in. diameter drawn wire and a design in reinforced concrete. The photograph was taken just before the deck was concreted. The bridge was fully described in this journal for February 1953.

# A Prestressed Precast Footbridge.

The footbridge shown during construction in *Fig.* 1 spans 61 ft. 8 in. and connects the headgear building to the pithead baths at Methley Savile colliery in the West Riding of Yorkshire. It was built during 1954 for the North Eastern Division of the National Coal Board.

A cross-section (*Fig.* 2) shows the main dimensions. The bridge comprises two prestressed precast beams spaced 8 ft. apart and carrying on their bottom flanges a deck consisting of precast slabs 3¾ in. thick with a 1¼-in. topping, and on their upper flanges precast frames connected longitudinally by precast beams supporting a roof of asbestos-cement sheets covered with bituminous felt. At the headgear building the ends of the main beams are supported on reinforced concrete rocker bearings; the other ends are simply supported. Each beam is prestressed by five cables each comprising twelve 0·2 in. high-tensile wires; in addition there are some small-diameter mild steel bars the main purpose of which is to support the cables during concreting and to resist tensile stresses during erection. All the loads on the structure are supported by these two beams. The imposed loads used in the design are 15 lb. per square foot for the roof and 100 lb. per square foot for the deck. The maximum compressive stress in a beam, under full load, is 1460 lb. per square inch, and a minimum compressive strength of 4000 lb. per square inch was required in the concrete before the cables were tensioned. The frames are joined to the top flanges of the beams by ¾-in. brass dowels, and the lintels are connected to the frames by being bolted to steel plates projecting from the sides of the frames (*Fig.* 3), the ends of the lintels being recessed for this purpose.

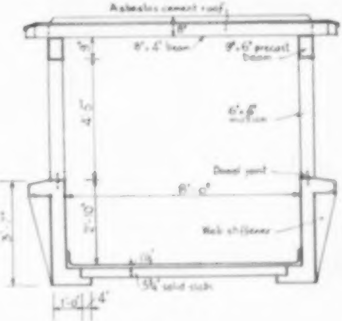



**Fig. 2.—Cross-section.**

All the parts for the bridge were precast in the contractor's yard and carried by road to the site. The main beams, 62 ft. 5½ in. long, were cast on a concrete base in timber shutters lined with plywood painted with a rubber-base paint. Each beam was cast in three parts, so that a shutter one-third of the length of a beam only was required, and this shutter was

**Fig. 1.—The Bridge during Erection.**

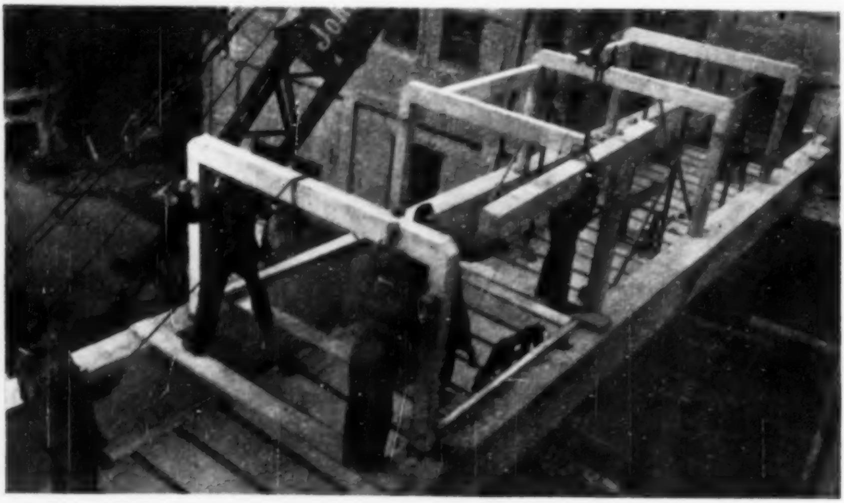


**Fig. 3.—Erecting the Frames.**

used six times. Succeeding parts of a beam were cast so that one end of the shutter was formed by the end of the part previously cast. The cables, encased in metal tubes, were placed in the shutters and supported by the mild steel stirrups while the concrete was placed and consolidated. A 1 : 1 : 2 mixture was used and the concrete was kept continually moist until crushing tests on cubes cast at the same time as a beam indicated that sufficient strength had been attained to enable the cables to be tensioned. All the cables in a beam were tensioned before the beams were moved.

The main beams, each of which weighs 11 tons, were carried to the site on two 12-tons trailers drawn by a tractor, the

**Fig. 4.—Lifting a Main Beam.**

ends of the beams resting on swivelling supports at the front of the first trailer and at the rear of the second trailer.

The bridge spans a road and a single-track railway siding separated from the road by a wall 8 ft. high; consequently movement on the site was restricted and two 12-tons cranes, one on the road and one on the railway siding, were used to lift the main beams (*Fig.* 4). Access to the site for any work which would interfere with traffic was confined to a week-end. However, on the Saturday on which work was to start a high wind prevented the main beams being hoisted on to their supports, some 27 ft. above ground, and all the work was done on the following day. Erection of the beams was done in 3¼ hours, and the lifting and fixing of the deck slabs, frames, and lintels required 9½ hours. Fixing of the asbestos-cement roof, the glazing between the frames, and other work could then be done from the bridge so that there was no interference with traffic below.

The work was carried out under the direction of Mr. J. A. Dempster, F.R.I.B.A., Divisional Chief Architect of the North Eastern Division of the N.C.B., and the associated architect, Mr. Eric Hooper, A.R.I.B.A. The structural design was by Messrs. Matthews & Mumby, Ltd., who made the precast members at their works in Manchester and erected the bridge. The Freyssinet system of prestressing was used.

# Bank Premises at Exeter.

THE building now under construction for Barclays Bank, Ltd., at Exeter occupies a corner site about 120 ft. by 150 ft. on sloping ground. Part of the site at the junction of the two streets is occupied by existing bank premises which will remain in use until the new accommodation is complete. The new building will also include shops, showrooms, and offices with basements below a four-story block fronting the two streets.

The new banking hall is 54 ft. diameter and is at the rear of the existing bank; access for vehicles to the basements is provided by a sunken service-road beneath the hall. The floor of the banking hall (*Fig.* 1) is supported at the centre on a column with a flared head, and near the circumference on two concentric rings of beams and columns. The outer columns (*Fig.* 2) extend through the banking hall to support a flat roof at first-floor level. The inner ring of columns rises 5 ft. above the roof and will support the ring-beam of a thin concrete dome 40 ft. in diameter.

The main staircases and lifts are to be contained within a shaft of 28 ft. diameter formed of load-bearing brickwork and a reinforced concrete frame. Two staircases of precast concrete steps will span between concentric brick walls, the inner of which will encircle the lift shaft. An

**Fig. 1.—Banking Hall Floor from Below.**

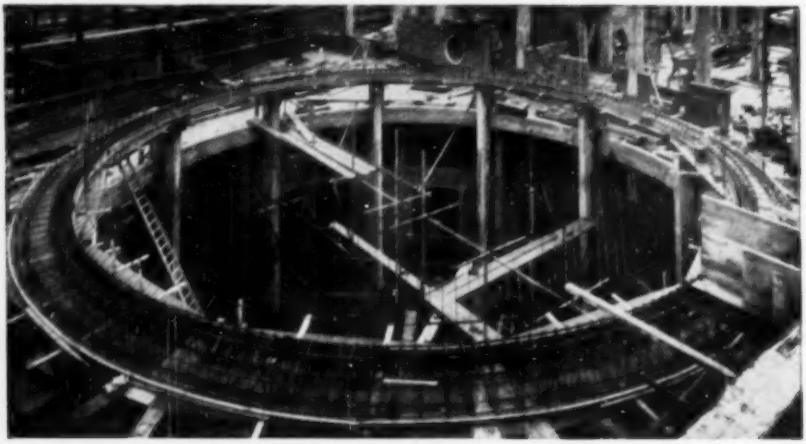


**Fig. 2.—The Ring Beam for the Dome.**

**Fig. 3.—A Construction Joint in a 6-ft. Deep Beam.**

inner frame of beams and columns will support the landings and lift-gear, and the main frame of the building will carry the outer brick wall at each floor.

The shop, showroom, and office accommodation is generally of slab and beam construction with columns spaced on a grid of about 26 ft. by 14 ft., but is designed to be constructed in two stages to permit the existing bank to be used during the first stage of construction. Various requirements precluded the use of double columns at the junctions between the two stages and limited the choice of positions for the construction joints. In one case (*Fig.* 3) a joint has been formed in a heavily-loaded beam 6 ft. deep and of 28 ft. span, 7 ft. from a column, and in several cases joints are to be made at the faces of columns.

The architects are Messrs. W. Curtis Green, R.A., Son and Lloyd. The consulting engineers for the structural work are Messrs. John F. Farquharson & Partners, for whom the shell dome was designed by Mr. C. V. Blumfield. The contractors are Messrs. John Garrett & Son, Ltd.

# Foundation Designed for Subsidence due to Mining.

In *Figs.* 1 and 2 are shown part of the foundation for a school at Heanor on a site liable to subsidence due to mining. The consulting engineers are Messrs. Ove Arup & Partners, who write as follows.

The structural problems created by building on sites liable to subsidence can be solved in two ways, namely (1) by designing the structure and its foundation, or the foundation only, to move with the settlement of the earth, or (2) to support the building in such a way that movement of the earth does not affect its stability.

In the first method either the whole structure may be designed to allow for settlement or the superstructure may be designed normally and the foundation only designed to allow for settlement. If the superstructure and the foundation are both designed to allow for settlement, the result is a continuous three-dimensional frame in reinforced concrete, and as the structure must be conceived primarily to allow for subsidence it is possible that the architectural planning may be restricted. If the foundation were designed to allow for settlement, a flexible raft foundation would be used and this would concentrate the protection outside the superstructure; this means that within reasonable limits the planning of the building is not restricted by the arrangement of the ribs of the raft, which can be determined from structural considerations only. A further factor, which is perhaps of over-riding importance, is that in any given case the forces and moments which have to be dealt with increase in direct proportion to the weight of the building to be supported. From this viewpoint, therefore, the weight of the building should be kept as small as possible. For two-story buildings, timber or light steel construction is to be preferred to reinforced concrete or load-bearing brickwork. In addition, an open-grid raft foundation should be used because it is lighter than a solid slab. A further advantage of the open-grid raft is that at the crests of the subsidence the ribs cut into the ground. This reduces the spans of the ribs and consequently the bending moments.

The second possibility, that is supporting the building so that settlement of the earth does not affect its stability, requires each unit to be supported at three points. As most buildings are square or rectangular in plan this results in considerable moments being developed. These moments are permanently present in the structure, and only normal working stresses can be used to resist them. In some types of buildings it may be possible for these moments to be resisted by the superstructure. However, if triangular units were a planning possibility, this line of approach would be worth further investigation.

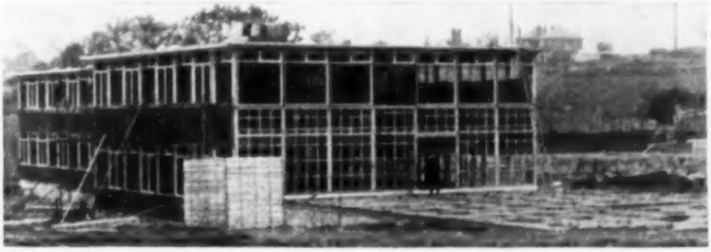


**Fig. 1.—Grid Raft on Right.**

In the case of this school it was decided to build the superstructure in timber, using the Derwent system, and concentrate the protection against subsidence in the foundations by using a series of open-grid rafts. The rafts are on or just below ground level on a 9-in. layer of pit-bind, which is intended to have a cushioning effect when the ground

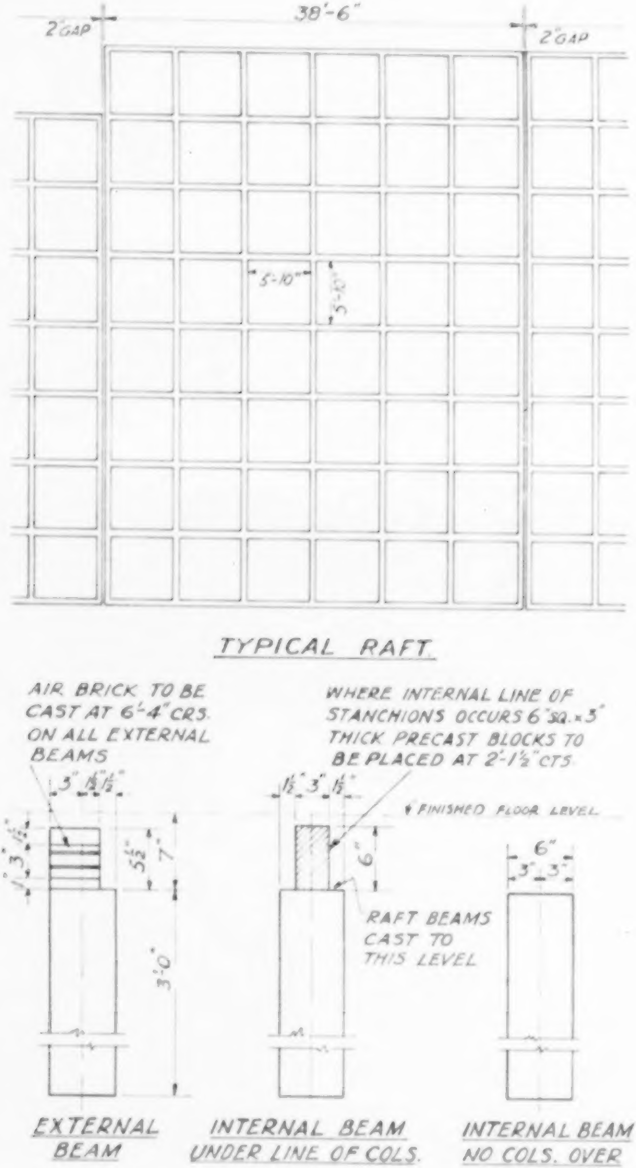



**Fig. 2.—Foundation designed for Mining Subsidence.**

settles locally. Joints are introduced between the raft units to limit the maximum dimension in any one direction to 60 ft.

The ribs of the raft are 3 ft. deep by 6 in. wide and are at 6 ft. 4 in. centres in both directions. The assumptions made in designing the ribs were: Compressive stress in 1 : 2 : 4 concrete, 2000 lb. per square inch; tensile stress in cold worked square twisted bars, 54,000 lb. per square inch; maximum positive bending moment, $0{\cdot}065pl^2$; maximum negative bending moment, $0{\cdot}125pl^2$.

The architect is Mr. F. Hamer Crossley, County Architect of Derbyshire, and the contractors Messrs. Vic Hallam (Contractors), Ltd.

# A Store at Coventry.

This building (*Fig.* 1), which was commenced in April, 1951, and opened in October, 1954, has five stories with a frame constructed entirely of reinforced concrete. Adjoining it, and extending below road level, is a basement of two stories, also of reinforced concrete, constructed within an old excavation of about 350,000 cu. ft. cut in rock which originally contained the basement of a building that was demolished during an air raid. A dual-carriageway road passes over the basement, which is designed to carry the road and the full Ministry of Transport loading.

The main superstructure is in two forms of construction. The sales area is of flat-slab construction supported by columns on a grid of 27 ft. 6 in. by 27 ft. 6 in., while two bays to one side are of beam-and-slab construction. The shape of the columns (*Fig.* 2) supporting the flat slabs was chosen for both æsthetic and functional reasons in that it encloses the standard

Fig. 2.

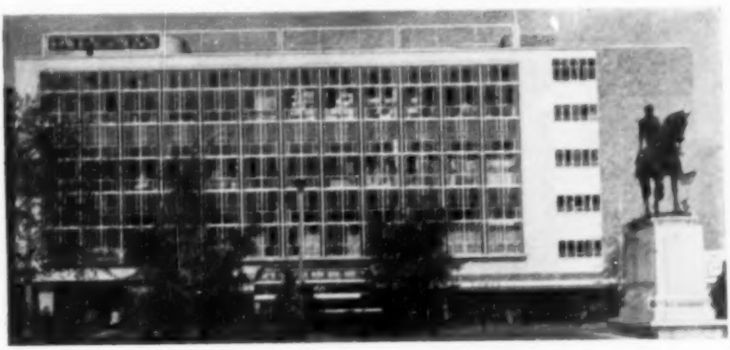


Fig. 1.

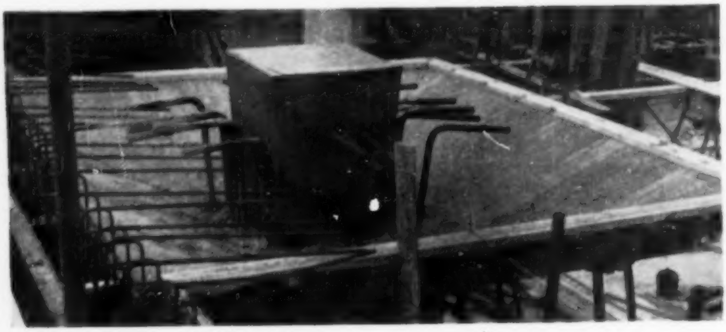


**Fig. 3.—Shuttering for Top of Column.**

column shaft, the conical column head, and the dropped slab of the dimensions required by B.S. Code of Practice No. 114. Pockets of sufficient depth to accommodate the bond length of the reinforcement (*Fig.* 3) were formed in the column head and dropped slab. Placing the column reinforcement in these pockets dispensed with the need for splice-lengths of reinforcement and avoided congestion of steel in the reduced section at the foot of the column.

Most of the walls are constructed of reinforced aerated concrete with a density of 90 lb. per cubic foot. The brick facing was used as external shuttering, and removable shuttering for the interior surfaces. This type of construction provides the insulation required and is also strong enough to resist the wind on an unsupported height of 15 ft. 9 in. between floors. The contractors found it to be a quick and convenient mode of construction.

The architects are Messrs. Rolf Hellberg, F.R.I.B.A., and Maurice H. Harris, A.R.I.B.A., and the consulting engineers Messrs. Scott & Wilson, Kirkpatrick & Partners. The contractors were Bovis, Ltd.

## Beauty and Utility.

THE following abstract from an address by Professor A. E. Richardson, President of the Royal Academy, on the subject "Can Craftsmanship Survive?", expresses in a forceful manner the views given in our Editorial Note this month, which was written before the President's speech was made. It also gives the views of an artist on opinions such as those that appear to be held by the Council of Industrial Design, given on page 70 of this number, that line, utility, and functionalism are all that are necessary in the design of lamp posts.

"Travelling through England," said Professor Richardson, "I think of the great crowd of people who put utility before beauty. They think that utility is everything and beauty is nothing. The idealists are far more important than the realists who give us nothing but stark realism. Idealism aims at beauty. It is what the people crave for, and it is our duty to see that they get it. Take no notice of the critics who say, 'Remain stark naked, tear off everything, strip-tease to the skeleton, the skeleton is what you are after.' It is not. It is the flesh, and the artist knows that. Nothing should be streamlined in ordinary things except the water-closet. There is talk of streamlining elevations, all for notoriety. I say, Have no mercy. Boycott the rubbish—shun it!"

# A Bus Garage in London.

## LONG-SPAN BOWSTRING AND BOX GIRDERS.

A BUS GARAGE (*Fig.* 1) at Shepherd's Bush, London, recently built for the London Transport Executive will accommodate 123 buses, and provision is made for an extension for another 25 buses. The positions of the supports of the main roof were largely determined by the irregular shape of the site and by the fact that an existing garage on the site had to be used while the new one was being built. The new garage comprises a parking area of about 45,300 sq. ft. with inspection pits, workshops, and stores, and an area of 6700 sq. ft. for servicing, oiling, cleaning, and washing buses; there are also locker rooms, drying rooms, a boiler house, and other ancillary rooms. An office and canteen block is a separate two-story structure.

The garage is in reinforced concrete with brick panel exterior walls supported on reinforced concrete ground beams. The roof over the parking area consists essentially of reinforced concrete hollow

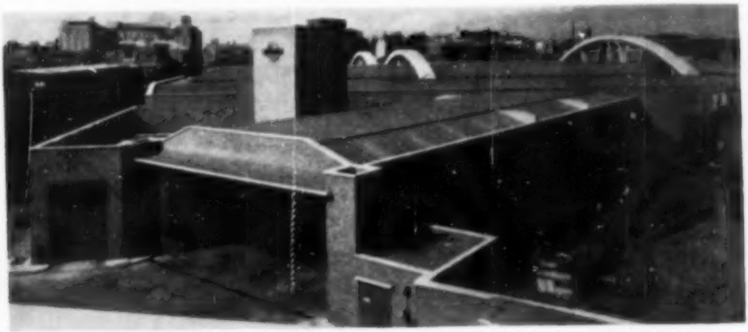


**Fig. 1.—General View.**

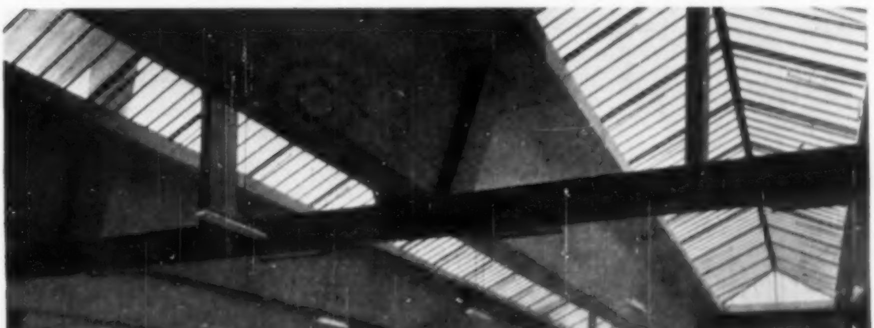


**Fig. 2.—Box Girders.**

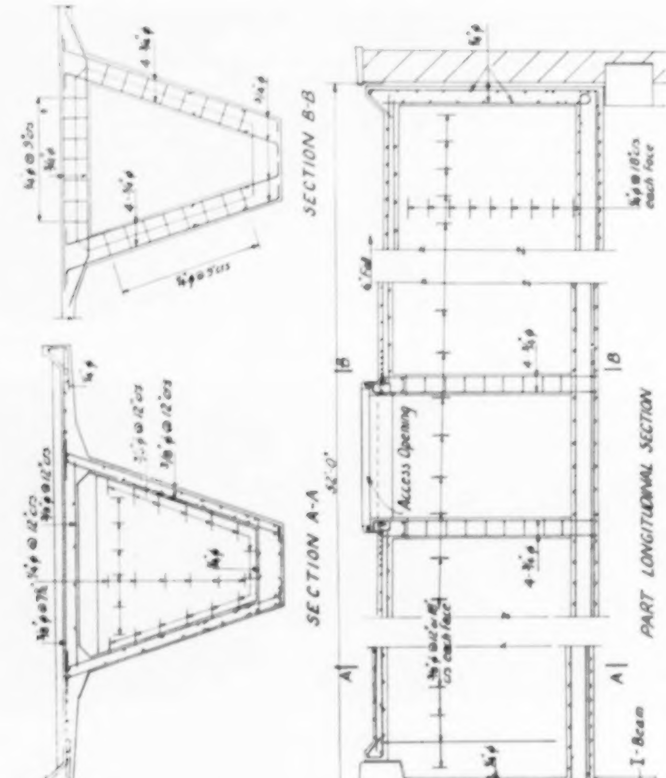



Fig. 3.—Details of a Typical Box Girder.

box-girders supported on main transverse members (*Fig.* 4) and comprises a deep fascia beam (*Fig.* 2) over the main entrance which incorporates a canopy and with which the box-girders are monolithic, two bowstring girders each 99 ft. long, a bowstring girder 132 ft. long, a deep I-shape beam 132 ft. long, and a number of beams and columns at the eastern end of the garage.

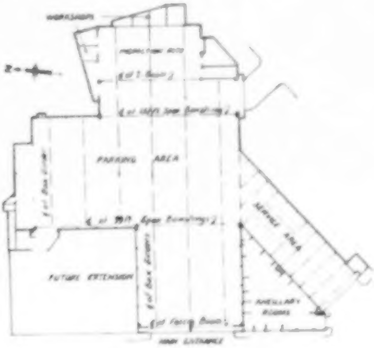



**Fig. 4.—Key Plan.**

## The Box-girders.

The beams and columns in the end wall support reinforced concrete box-girders of 52 ft. span at 33 ft. centres. These girders (*Figs.* 2 and 3) are 8 ft. deep and 8 ft. wide with a compression flange 16 ft. wide and an average thickness of 5 in. The bottom is 3 ft. wide and 1 ft. thick and the sides are 6 in. thick. There are twin transverse stiffening ribs in the girders, at the middle of the shorter spans and at about the third-points of the longer spans. In the end walls there are openings which allow men to pass from one span to another. Access to the girders is provided through the top, which forms part of the roof, and services are accommodated in the girder. Pitched roof-lights are provided between the edges of the flanges of adjacent girders; they are 17 ft. wide and consist of glazing bars between a ridge-piece, which is supported on steel trusses at intervals, and the edges of the girders. The roof-lights over the central portion of the parking area are 272 ft. long.

The girders of 52 ft. span are supported

**Fig. 5.—Staging and Shuttering for a Bowstring Girder.**

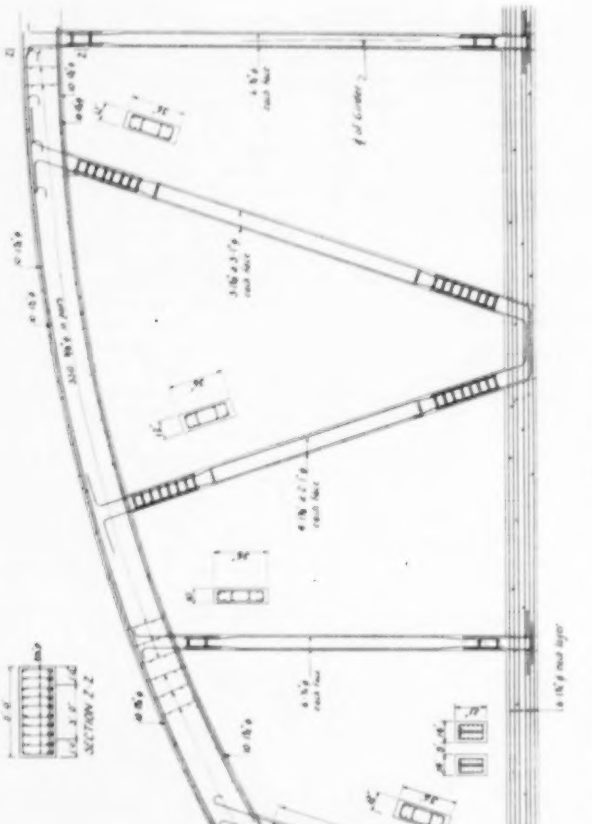



**Fig. 6.—Detail of Bowstring Girder of 132-ft. Span.**

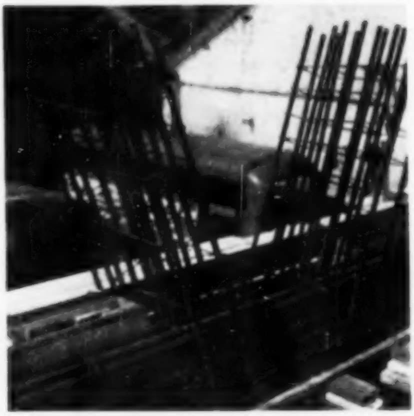


**Fig. 7.—Bars at the Junction of the Main Tie and Hangers.**

at one end on an I-section beam 132 ft. long which is divided into spans of 99 ft. and 33 ft. This beam is 10 ft. deep and comprises a bottom flange 4 ft. 9 in. wide by 2 ft. 3 in. deep, a web 15 in. thick, and an upper flange 4 ft. wide by 1 ft. 6 in. deep. The concrete was a 1 : 1½ : 3 nominal mixture. The 99-ft. span was cast first and its temporary supports removed before the 33-ft. span was constructed. A large negative bending moment over the interior support was thus avoided and it was possible to maintain a uniform cross section throughout the beam. This beam also supports four 34-ft. 6-in. box-girders, three of which are on the same centre-lines as the 52-ft. span girders.

### The Bowstring Girders.

The other ends of the 34-ft. 6-in. girders are carried on a bowstring girder of 132 ft. span (*Figs.* 5 and 6). This girder has a rise of 26 ft. 4 in. and also supports the

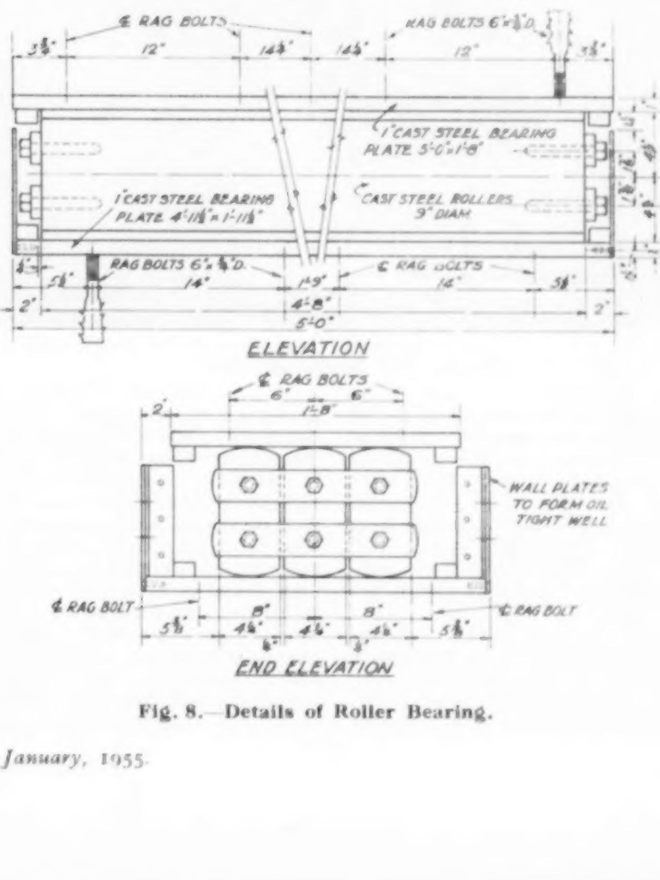



**Fig. 8.—Details of Roller Bearing.**

ends of four box-girders 108 ft. long. The superimposed load on the bowstring girder consists of four concentrated loads of 128 tons each; the horizontal thrust at each springing is 437 tons and the total vertical load on each column is 357 tons. The arched rib is 5 ft. wide by 2 ft. 3 in. deep at the crown increasing to 3 ft. at the springing. Except at the ends, the horizontal ties (*Fig.* 6) consist of twin members 1 ft. 9 in. deep by 1 ft. 2 in. wide each containing eighteen 1½-in. diameter bars. The members are united at the ends of the girder to provide sufficient width to accommodate twelve additional 1½-in. diameter bars required to anchor the arched rib. The main bars are 45 ft. long and joined by double-Vee butt welds. The hangers are 3 ft. by 1 ft. and, in order to prevent the concrete from cracking when the girder was loaded, temporary gaps were left in the concrete. After the main bars had extended under load these gaps were filled.

To prevent bending moments being transferred to the supporting columns, a hinge is provided at one end of the girder and a roller bearing at the other. The bearing (*Fig.* 8) consists of 9-in. diameter cast steel rollers between two cast steel plates 1 in. thick; the plates are connected to the column by ragbolts and the top plate is similarly secured to the underside of the girder. The whole bearing is enclosed by thin metal plates and is filled with lubricant.

Parallel to this girder and 108 ft. apart from it are two bowstring girders which are supported on a column at their adjoining ends and on roller bearings at the other ends. These girders are 99 ft. long and have a rise of 20 ft.; they are similar in construction to the bowstring girder of 132 ft. span. One of these bowstring girders supports the ends of three of the box-girders that span between it and the 132-ft. bowstring girder and three box-girders that span between it and the fascia beam over the entrance. The other bowstring girder at present supports one end of the box-girders spanning between it and the 132-ft. span bowstring girder as well as the ends of two box-girders 108 ft. long which at their other end are supported on beams and columns in the wall of the northern side of the parking area. This bowstring girder will carry additional loads when

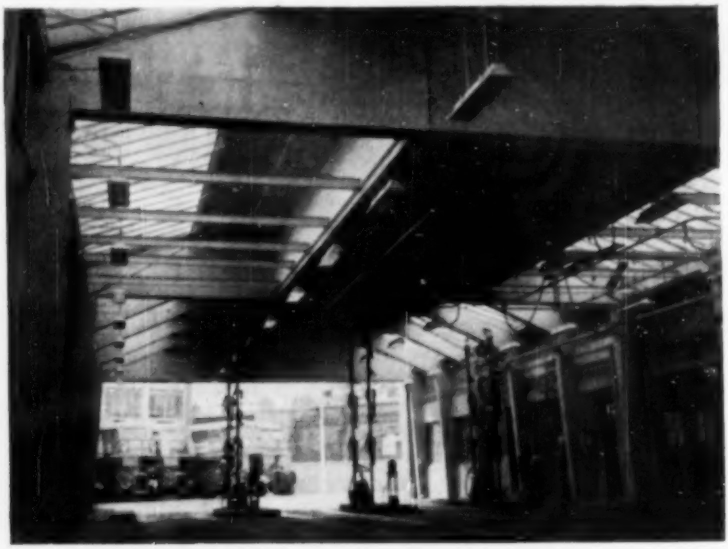


**Fig. 9.—The Servicing Area.**

the garage is extended. The roof at the northern edge of the parking area is supported by a half-box girder cantilevered from columns at the side which in turn are supported on cantilevered foundations designed to avoid placing the foundations below adjoining property. There are seventeen box-girders at present, and there will be twenty when the garage is extended.

Where the box-girders are supported by the I-beam or by the bowstring girders the bearing surfaces are separated by lead-cored sheeting and at the sides of the hangers by thick building paper. The gaps between the ends of the box-girders are filled with a flexible material to allow longitudinal movement.

The roof of the servicing area is supported by frames (*Fig.* 9) of 48 ft. 3 in. span at 13 ft. 6 in. centres, but where the washing plant, which is to be accommodated in the roof, occurs the frames are farther apart. The washing plant is to be supported on steel joists spanning between the walls and a reinforced concrete beam spanning between the frames.

The roofs were designed for a superimposed load of 30 lb. per square foot and the stresses used in the design were in accordance with the London County Council's by-laws. All the roofs are covered with asphalt, and thick bituminous felt was used to form the flashings between the bowstring girders and the box-girders; expansion rolls were formed in the flashings to allow for movement due to temperature changes.

The office and canteen building is a reinforced concrete framed structure of two stories. The frames are 46 ft. wide and are at 10 ft. 4½ in. centres. The exposed concrete of this building was made of white Portland cement, Leighton Buzzard sand, and crushed Darley Dale stone, mixed in the proportions of 1 : 1½ : 3; the surfaces were polished with carborundum discs.

The architects were Messrs. Adie, Button & Partners in association with Mr. Thomas Bilbow, F.R.I.B.A., Architect of the London Transport Executive; the consulting structural engineer was Mr. A. E. Beer, A.C.G.I., M.I.Struct.E., and the main contractors Messrs. Charles Booth & Son.

## Standardisation of Structural Members.

WITH a view to reducing the cost of shuttering, the British Standards Institution has issued B.S. No. 2359, "Preferred Dimensions of Reinforced Concrete Structural Members." The recommended dimensions are as follows.

Cross-sectional dimensions of all members.—6 in. rising in multiples of 1½ in. to 15 in. and thereafter in multiples of 3 in.

Slabs.—Length and width in multiples of 3 in.

Beams.—Length in multiples of 3 in.

Column bases and foundation blocks.—All dimensions in multiples of 3 in.

Square piles.—Sides 10 in. in increments of 2 in. to 18 in. Splays at corners, 1½ in.

Splays at junction of walls and floor of tanks and at junction of wall and base of retaining walls.—Angle 45 deg.; distance from wall and base, a multiple of 2⅛ in. so that the dimension of the slope will be 3 in. or a multiple of 3 in.

Column heads in flat-slab construction.—Splays 45 deg.; depth of splays in multiples of 2⅛ in., but 12¾ in. minimum; depth of thickening of slabs in multiples of 1 in. but 2 in. minimum; width of thickened slabs in multiples of 6 in.; distance between column heads in multiples of 3 in.

## "Retention Money."

A COMMITTEE appointed by the Minister of Works to consider the possibility of the present system of "retention money" on civil engineering contracts being replaced by bonds or guarantees has now issued its report ("Retention Moneys on Building and Civil Engineering Contracts." H.M.S.O. Price 6*d*.). The committee does not recommend such a change mainly on the ground of the undesirability of a third party having a financial interest in any dispute that may arise due to a claim for defective work or the inability of a contractor to complete the work. It is, however, suggested that guarantees may have some value in the case of work done by subcontractors, that the sum retained be as small as possible in relation to the work, and that moneys due to contractors should be paid as soon as possible.

# Sea Defences on the Norfolk Coast.

THE construction of sea defences along the Norfolk coast at Sea Palling included the construction of a reinforced concrete stepped wall (*Fig.* 1) 2½ miles long with a curved parapet 6 ft. high; the total height is about 12 ft.; the width is 17 ft. A line of steel sheet piling extends the full length of the wall to form the toe. The quantity of concrete used exceeded 25,000 cu. yd.; the reinforcement included 230 tons of bars and 40,000 sq. yd. of fabric.

The wall was designed for construction in lengths of 25 ft. and in six stages, each stage forming a stable structure giving an

**Fig 1.—The Wall.**

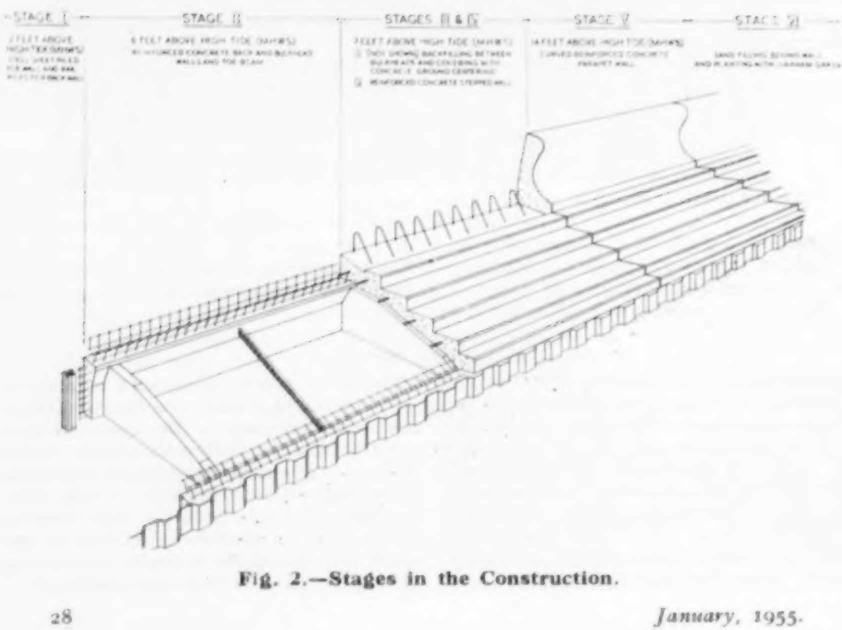



**Fig. 2.—Stages in the Construction.**

increasing measure of protection from flooding. The six stages of construction were (see *Fig.* 2): (1) Driving steel sheet piles at the toe and piles (formed of second-hand bull-head rails bolted together in pairs) in the back wall; (2) concreting the toe, back and bulkhead walls; (3) filling the space between the bulkhead walls; (4) constructing the stepped part of the wall; (5) constructing the curved parapet; (6) filling behind the wall. Joints in the wall, at 25 ft. centres, were formed with dowel bars and an expansion jointing material with a vertical sealing strip. For constructional purposes the work was divided into six sections with independent teams of workmen carrying out the successive stages on each section.

The shutters (*Figs.* 3 and 4) for the curved parapet wall were provided with wheels so that after they were eased away from the finished wall they could readily be moved to the next section, the previously constructed steps providing a base for this operation.

A 34E Koehring twin-batch machine in a static position was used (*Fig.* 5) for weighing the materials. The cement was delivered loose and the storage capacity was 190 tons. A hopper was provided for the aggregate. Measured quantities of materials were supplied to the mixer and the batch of concrete was taken by the container on the boom to an elevated hopper from which it was delivered to high-discharge lorries for distribution to the parts of the wall being concreted; twelve lorries were used, one for supplying each section. Concreting was carried

**Fig. 4.—The Shuttering for the Parapet.**

**Fig. 3.—Moving the Shuttering.**


**Fig. 5.—The Mixing and Batching Plant.**

out at the rate of 300 cu. yd. a day. The concrete was a nominal 1 : 2 : 4 mixture and was consolidated with immersion vibrators. The aggregates were washed pit sand and gravel.

The wall was constructed for the East Suffolk and Norfolk River Board. The consulting engineers were Messrs. Lewis & Duvivier, and the contractors Messrs. John Laing & Son, Ltd.

## Workshop at a Laboratory.

THIS is a small workshop at the laboratory at Wexham Springs of the Cement and Concrete Association. The span of the roof is 50 ft., and the shells are prestressed in the longitudinal direction by high-tensile steel bars of $\frac{5}{8}$ in. and $\frac{3}{4}$ in. diameter. As an experiment, the roof was not waterproofed in any way. A nominal mixture of $1{:}1\frac{1}{2}{:}3$ with aggregate of $\frac{3}{4}$ in. maximum size was used and copper water-bars were inserted in the longitudinal construction joints. The roof was watertight six months after the centering was removed. The columns at each end of the building are tied together by prestressing cables in the beams at mid-height. The architect is Mr. W. R. Oram, A.R.I.B.A. The consulting engineers are Messrs. Ove Arup & Partners, and the contractors Messrs. Holland & Hannen and Cubitts, Ltd.

# Bending and Axial Forces.

THE following notes on a method of calculating the stresses in a member subjected to bending and axial forces have been received from Mr. R. J. Bartlett, A.M.I.Struct.E.

### Members Subjected to Bending and Direct Compression.

EQUAL REINFORCEMENT IN COMPRESSIVE AND TENSILE SIDES.—A member (*Fig.* 1) is subjected to a bending moment and a direct force such that tensile stresses are developed in one face. The notation is as shown except that $A$ is the total cross-sectional area of steel in the member.

If the load and bending moment are such that tensile stresses are produced the stresses may be calculated when the value of $n$ is known. Assume a value of $n$ and substitute it in

$$\frac{2n^3 + 3an^2 + 3c}{3(n^2 + 2an + 2b)} \quad . \quad . \quad (1)$$

where $a = e - \frac{D}{2}$, $b = \frac{mA}{B}e$,

and $c = \frac{mA}{B}\left(eD + \frac{r^2}{2}\right)$;

$m$ is the modular ratio. Evaluation of (1) gives a corrected value of $n$, and if this new value differs only slightly from that assumed it may be accepted. If the difference is appreciable the new value should be corrected by insertion in (1), and the procedure repeated until the value inserted in (1) agrees with that resulting from the evaluation of the expression. Usually two corrections only are necessary.

The corrected value of $n$ may then be used to calculate the compressive stress $c_c$ in the concrete and the tensile stress $t$ in the steel from

$$c_c = \frac{2W}{Bn + 2mA - \frac{mDA}{n}} \quad . \quad (2)$$

$$t = \frac{mc_c(d - n)}{n} \quad . \quad . \quad . \quad . \quad (3)$$

The derivation of (1) is given in Appendix 1.

EXAMPLE 1.—A column 12 in. by 9 in. overall, and reinforced with two ⅞-in. bars in both the compressive and tensile sides, is subjected to a direct load of 22,500 lb. and a bending moment of 150,000 in.-lb. Calculate the maximum stresses if $r = 9$ in., $d = 10{\cdot}5$ in., and $m = 15$.

The eccentricity $e$ is $\frac{150{,}000}{22{,}500} = 6{\cdot}667$ in.

$$a = 0{\cdot}667,\ b = 26{\cdot}8,\ c = 485.$$

Assume that $n$ is 6·5 in. Then (1) = 6·675 in., which is sufficiently near to the value assumed to be acceptable. Then, from expressions (2) and (3), $c_c = 670$ lb. per square inch, and $t = 5760$ lb. per square inch.

REINFORCEMENT IN TENSILE SIDE ONLY.—In this case the maximum allowable tensile stress in the steel may be used, and the area of steel and the compressive stress in the concrete calculated.

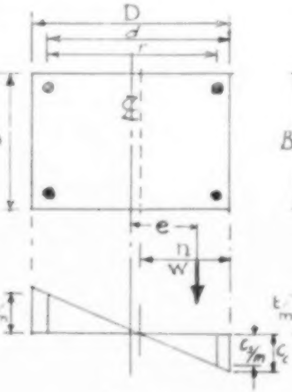



Fig. 1.

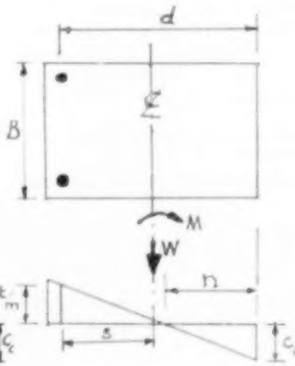



Fig. 2.

The procedure is similar to that previously described except that the assumed value of $n$ is substituted in

$$\frac{2n^3 - 3dn^2 - 6kd}{3(n^2 - 2dn - 2k)} \quad . \quad . \quad (4)$$

where $k = \dfrac{(M + Ws)m}{Bt}$ (*Fig.* 2). The corrected value of $n$ is then substituted in expressions (5) and (6) to obtain the compressive stress $c_e$ in the concrete and the area $A$ of steel.

$$c_e = \frac{6(M + Ws)}{Bn(3d - n)} \quad . \quad . \quad (5)$$

$$A = \frac{cBn - 2W}{2t} \quad . \quad . \quad . \quad (6)$$

EXAMPLE 2.—A member with a cross section 12 in. by 9 in. is subjected to a thrust $W$ of 22,500 lb. and a bending moment $M$ of 150,000 in.-lb. If $t$ is 18,000 lb. per square inch, $d$ is 10·5 in., $s$ is 4·5 in., and $m$ is 15, calculate $c_e$ and $A$.

$M + Ws = 251{,}250$ in.-lb., $k = 23{\cdot}265$. Assume that $n$ is 5·5 in. The corrected value of $n$, obtained from (4), is 5·28 in., which is acceptable. Therefore from (5) and (6) $c_e = 1188$ lb. per square inch, and $A = 0{\cdot}317$ sq. in.

## Members Subjected to Bending and Direct Tension.

EQUAL REINFORCEMENT IN COMPRESSIVE AND TENSILE SIDES.—The procedure is as described for members subjected to direct compression except that $-e$ is substituted for $e$ in expression (1), and $-W$ for $W$ in (2) and (3).

REINFORCEMENT IN TENSILE FACE ONLY.—The procedure is as described for members subjected to direct compression except that $-W$ is substituted for $W$ in expressions (4), (5), and (6).

## Appendix 1.—Derivation of Expressions (1) and (4).

From the geometry of the stress diagram (*Fig.* 1),

$$\frac{c_e}{n} = \frac{c_s}{m\left(n - \dfrac{D - r}{2}\right)} = \frac{t}{m\left(\dfrac{D + r}{2} - n\right)}.$$

The total tensile force in the steel $+ W$ = the total compressive force in the steel and concrete.

Therefore $\dfrac{At}{2} + W = \dfrac{Bc_e n}{2} + \dfrac{Ac_s}{2}$.

The moment of $W$ and the total compressive force, calculated from the centre of the steel in tension, is zero. Therefore

$$W\left(e + \frac{r}{2}\right) = \frac{Bc_e n}{2}\left(\frac{D + r}{2} - \frac{n}{3}\right) + \frac{A}{2}c_s r.$$

From the foregoing equations $n$ may be obtained:

$$n^3 + 3n^2\left(e - \frac{D}{2}\right) + \frac{6mAe}{B}n - \frac{3mA}{B}\left(eD + \frac{r^2}{2}\right) = 0,$$

which may be written in the form

$$n^3 + 3an^2 + 6bn - 3c = 0 \quad . \quad (7)$$

Similarly, from *Fig.* 2, $n$ may be expressed as

$$n^3 - 3dn^2 - \frac{6m(Ws + M)}{Bt}n + \frac{6md(Ws + M)}{Bt} = 0$$

or $\qquad n^3 - 3dn^2 - 6kn + 6kd = 0 \quad . \quad (8)$

The foregoing functions of $n$ may be expressed as $f(n) = 0$, and if the assumed value of $n$ is incorrect then $f(n)$ will not equal zero but $f(n) + \Delta n = 0$ where $\Delta n$ is the amount by which $n$ differs from the actual value. From Taylor's theorem

$$f(n) + \Delta n = f(n) + \frac{\Delta n df(n)}{dn} + \frac{\Delta n^2}{2}\frac{d^2f(n)}{dn^2}.$$

If $\Delta n$ is small compared with $n$, then $\Delta n^2$, $\Delta n^3$, etc., may be ignored; and

$$0 = f(n) + \frac{\Delta n df(n)}{dn},$$

and $\qquad \Delta n = -\dfrac{f(n)}{\dfrac{df(n)}{dn}}.$

The actual value of $n$ is then

$$n + \Delta n = n - \frac{f(n)}{\dfrac{df(n)}{dn}},$$

and from (7) the actual value of $n$ is

$$n - \frac{n^3 + 3an^2 + 6bn - 3c}{3(n^2 + 2an + 2b)} = \frac{2n^3 + 3an^2 + 3c}{3(n^2 + 2an + 2b)}.$$

The inaccuracy due to using $m$ instead of $(m - 1)$ for the steel in compression may be avoided by adding $+\dfrac{A(r - 2e)}{4B}$ to the value of $b$, and $+\dfrac{A(D - r)\left(\dfrac{r}{2} - e\right)}{2B}$ to the value of $c$ in expression (1). Usually this correction affects the stresses by between 2 and 3 per cent.

## Members Subjected to Two Bending Moments and a Direct Load.

The following notes set out a method of analysing fairly accurately the stresses in reinforced concrete columns in which tensile stresses are present due to two bending moments at right angles to each other in addition to an axial load. The analysis applies to rectangular columns reinforced equally in all corners with the axes of symmetry in the planes of the bending moments (see *Fig.* 3). Most corner columns in a frame satisfy these conditions.

First it must be decided which of the two moments, when combined with the axial load, will produce the greater tensile stresses. The position of the neutral axis and the maximum stresses are then calculated for the axial load $W$ and this moment, say $M_1$. The accuracy of the subsequent calculations depends to a large extent on the accuracy with which they are computed.

The second part of the analysis is to determine what effect $M_2$ has on the stresses due to $W$ and $M_1$. Calculate

$$K = \frac{Am}{Bn}, \quad P = 3Ks^2, \quad G = 3c_1(K+1),$$

$$N = \frac{4M_2}{GB^2n}, \quad R = \frac{4(1 + 2K - P)}{N},$$

$$T = \frac{1+P}{3N}, \text{ and } \alpha = \frac{1}{8T}$$

The secondary neutral axis in the section due to $M_2$ is close to the centre-line. Its distance from the centre-line is greatest when $M_2 = M_1$ and diminishes as $M_2$ diminishes. An approximate value for this distance, expressed as a ratio of $B$, is given by $\alpha$. A more accurate value is given by

$$\frac{0{\cdot}125 - 3\alpha^2 - 2R\alpha^3 - 54\alpha^4}{T - 6\alpha - 3R\alpha^2 - 72\alpha^3} \quad . \quad (9)$$

The range of variation for this adjusted $\alpha$ is from zero to about 0·045 in most cases. If $\alpha$ is less than 0·025 the term $54\alpha^4$ may be omitted from the expression, and if it is less than 0·01 the terms $2R\alpha^3$ and $72\alpha^3$ may also be omitted. The maximum stress in the concrete due to $M_2$ is now given by

$$c_2 = GL \quad . \quad . \quad (10)$$

where $L$ is read from *Fig.* 4.

If $\alpha$ is considered to be sufficiently accurate without adjustment, $R$ need not be calculated.


**Fig. 3.**

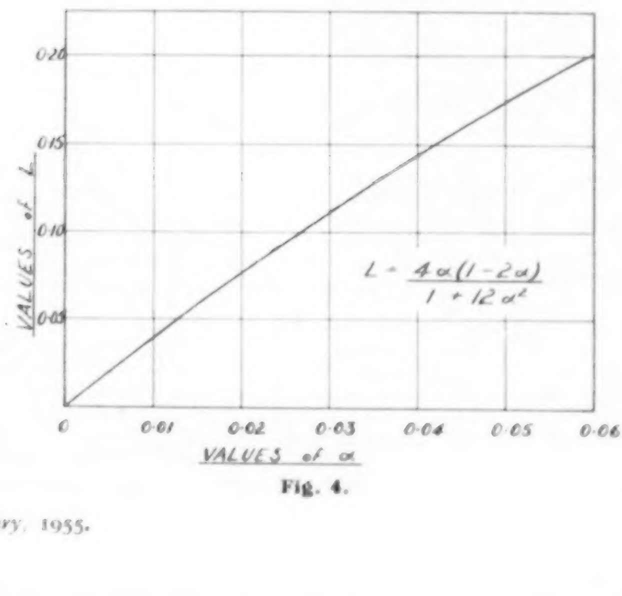



**Fig. 4.**

The maximum stress in the steel due to $M_2$ is obtained from

$$t_3 = c_2 m \frac{s + 2\alpha}{1 - 2\alpha}. \qquad (11)$$

The maximum stress in the concrete at corner A is the total of $c_2$ and $c_1$ calculated for $W$ and $M_1$. The maximum tensile stress in the steel will occur in the bar nearest to corner C and is obtained by adding $t_3$ and $t$ calculated for $W$ and $M_1$.

EXAMPLE 3.—With the section bending moment and load given in Example 1, investigate the stresses when an additional moment $M_2 = 50{,}000$ in.-lb. is applied at right angles to $M_1$.

$$K = \frac{2{\cdot}41 \times 15}{9 \times 6{\cdot}675} = 0{\cdot}602\,; \quad P = 0{\cdot}802\,;$$

$$G = 3 \times 670 \times 1{\cdot}602 = 3220\,;$$

$$N = \frac{4 \times 50{,}000}{3220 \times 81 \times 6{\cdot}675} = 0{\cdot}115\,;$$

$R = 48{\cdot}75$; $T = 5{\cdot}224$; $\alpha = 0{\cdot}024$.

Adjusted $\alpha = 0{\cdot}02443$.

From *Fig.* 4, $L = 0{\cdot}0923$,

$c_2 = 3220 \times 0{\cdot}0923 = 297$ lb. per square inch,

$t_3 = 3360$ lb. per square inch.

The maximum stress in the concrete is $670 + 297 = 967$ lb. per square inch and the maximum stress in the steel is $5760 + 3360 = 9120$ lb. per square inch.

## Appendix 2.—Derivation of Expressions (9), (10), and (11).

In *Fig.* 5 ABCD is the cross section of the column. The variation of stress due to $W$ and $M_1$ is represented by vertical ordinates of plane A′B′C′D′ with respect to ABCD, and the line EF common to both planes is the primary neutral axis found for $M_1$ and $W$. The vertical ordinates of plane A″B″C″D″ with respect to A′B′C′D′ give the variation of stress due to $M_2$, and line E′F′ (that is E″F″ projected on ABCD) is the secondary neutral axis. The final neutral axis due to $M_1$, $M_2$, and $W$ is E‴F‴ and is the line common to planes ABCD and A″B″C″D″.

The following deals solely with the effect of $M_2$ on the stresses caused by $W$ and $M_1$. Let

$lB$ be the distance of E′F′ from edge AD.

$\alpha B$ the distance of E′F′ from centre-line of the cross section.

$c_4$ the maximum compressive stress in the concrete along edge AD.

$c_3$ the maximum loss of compressive stress in the concrete along edge CB.

$t_2$ the additional compressive stress in the bar at corner A = loss of tensile stress in the bar at corner D.


Fig. 5.

$t_3$ the loss of compressive stress in the bar at corner B = additional tensile stress in the bar at corner C.

By similar triangles, and by considering A′A″F″B″B′,

$$\frac{c_2}{l} = \frac{t_2}{m\left(l - \frac{1-s}{2}\right)} = \frac{t_3}{m\left(\frac{1+s}{2} - l\right)} = \frac{c_3}{1-l}.$$

As the additional compression plus the loss of tension equals the loss of compression plus additional tension,

$$\frac{c_2 nlB}{2} \text{ (prism A′A″F″OEX)} + \frac{c_2{}^2 nlB}{6c_1} \text{ (pyramid OEXE‴)} + \frac{A.t_2}{4} \text{ (bar at A)} + \frac{A.t_2}{4} \text{ (bar at D)}$$
$$= \frac{c_3 n(B - lB)}{2} \text{ (prism B′B″F″OFY)} - \frac{c_3{}^2 n(B - lB)}{6c_1} \text{ (pyramid OFYF‴)} + \frac{A.t_3}{4} \text{(bar at B)} + \frac{A.t_3}{4} \text{(bar at C)} \quad (12)$$

On substitution for $c_3$, $t_2$ and $t_3$, this reduces to $c_2 = 3c_1(K + 1) \times \dfrac{l(1 - 2l)}{1 - 3l + 3l^2}$.

Since $lB$ is less than half of $B$ by only a small amount it is sufficiently accurate to substitute $(\frac{1}{2} - \alpha)$ for $l$ in (12). Then

$$c_2 = G \times \frac{4\alpha(1 - 2\alpha)}{1 + 12\alpha^2} = GL.$$

The graph in *Fig.* 4 represents the relationship between $\alpha$ and $L$.

The couple set up by the internal forces in equation (12) is $M_2$, and by calculating the moments about the secondary neutral axis a fourth degree equation is found in terms of $l$, and on substituting $(\frac{1}{2} - \alpha)$ for $l$ it reduces to

$$18\alpha^4 + R\alpha^3 + 3\alpha^2 - T\alpha + 0{\cdot}125 = 0.$$

Expression (9) is derived from this equation. If in calculating the stresses due to $W$ and $M_1$ the area of concrete displaced by steel in compression is not included, $(m - \frac{1}{2})$ should be substituted for $m$ in calculating $K$.

The method described applies only if the final neutral axis cuts sides AD and CB which is the case if $c_2$ is less than $\dfrac{c_1(D - n)}{n}$. If $c_2$ is greater than this, then part of the pyramid OEXE‴ will be outside the section. If a column is subjected to axial load and one bending moment in a vertical plane that is not parallel to one of the axes of the section, the bending moment should first be resolved into two components on the axes of symmetry and treated as for $M_1$ and $M_2$.

## Congress on Prestressed Concrete.

The second international congress of the Fédération Internationale de la Précontrainte is to be held in Amsterdam from August 29 to September 2, 1955. Details are obtainable from Ir. J. A. H. Hartmann, Groningsestraat 15, La Haye, Holland.

## Coloured Concrete Surfaces.

The process of gluing special aggregates, generally to form a pattern, to a sheet of paper, which is placed in the bottom of a mould so that the face aggregate adheres to the backing concrete, has been used for many years in the precast products industry. In the United States this process is being applied to the cast-in-situ walls of structures, and is known as the "aggregate-transfer" method. The special coloured aggregates are glued to sheets of thin plywood which are placed in the shuttering so that the face aggregates are bonded to the structural concrete and the plywood is removed when the adhesion is destroyed by the water in the concrete. The surface thus transferred to the structural concrete is finished by wire-brushing, sand-blasting, or grinding.

# Residential Flats, Westminster.

THE construction of the reinforced concrete foundations, frames, and floors of eight blocks of flats in Warwick Way has recently been completed for the Westminster City Council. The sub-soil is sand and gravel on which the permissible pressure does not exceed 1·4 tons per square foot and the foundations are either piles or piers. The walls vary in thickness from 5 in. to 12 in., and the floors, which are designed for a superimposed load of 40 lb. per square foot, are mainly 4½ in. thick. The buildings are six stories in height with a basement; part of the basement is strengthened to serve as an air-raid shelter, and in two cases additional height is provided for recreational purposes.

Two electrically-operated tower cranes, one with a reach of 82 ft. and a maximum lifting height of 98 ft. 6 in. (*Fig.* 1), and a smaller one with a reach of 52 ft. 6 in. and a maximum lifting height of 65 ft. 7 in., were used to lift all the precast members, shuttering, reinforcement, and concrete. All the beams and stairs were precast in nests of moulds (*Fig.* 2), and the concrete consolidated by internal vibrators. The shutters for the walls enabled the concrete to be placed in lifts 9 ft. high. The floor centering was a steel telescopic type.

Fig. 1.

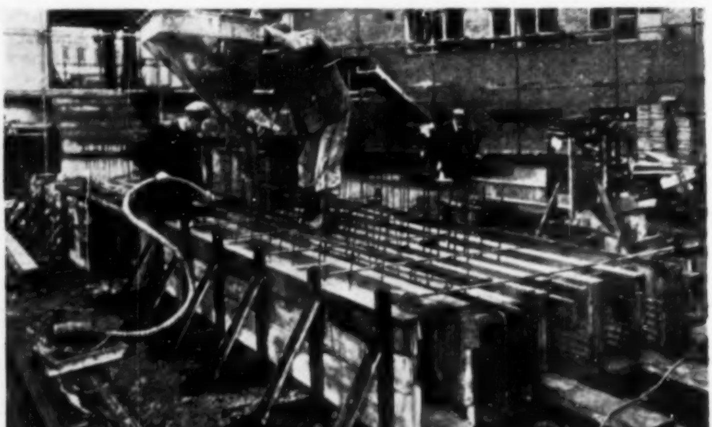


Fig. 2.

After the columns had been concreted in situ, the precast beams were placed in position on steel brackets fixed to the heads of the columns. The floor centering spanned between the beams. A story of each block was completed in ten to twelve working days.

The Westminster City Council's Housing Director is Mr. E. J. Edwards, A.R.I.B.A., and the architects are Messrs. Riches & Blythin (Mr. L. C. Holbrook, F.R.I.B.A.). The reinforced concrete was designed by Messrs. W. V. Zinn & Associates, consulting engineers. The main contractors were Messrs. J. Gerrard & Son, Ltd., and the contractors for the reinforced concrete work were Messrs. Rush & Tompkins, Ltd.

# A Factory at Bracknell, Berkshire.

A FACTORY (*Fig.* 1) erected for Hunter Chenilles, Ltd., comprises precast concrete frames and brick cavity walls. The roof is formed with precast slabs and glazing arranged to admit as much light as possible from the east and west. In order to disperse the light and conserve heat the glazing on the western slopes of the roof is opaque and is formed by two thicknesses of glass with glass fibre between them.

The frames (*Fig.* 2) are at 13 ft. 6¼ in. centres and have spans of 41 ft. 2 in. Five members are used to form the top of one three-bay frame; the outer members consist of a post and part of the roof and are joined to the other roof members of the outer bays by scarfed joints at the points of contraflexure of the bays. The roof members of the central bay are joined to those of the end bays at the valleys, where the ends of the roof members fit into recesses 1½ in. deep in the sides of the longitudinal beams, and rest on steel channels which project beyond the face of the beams. A high-tensile steel rod (*Fig.* 3), threaded at each end and fitted with nuts and washers to distribute the load on the concrete, causes partial continuity of the members when the nuts are tightened. The joint is grouted through a hole in the top of the beam, and a pad of grout is placed on the top of the beam to provide a seating for the gutter.

Internal columns at 27-ft. 1-in. centres support the longitudinal beams and are cast with part of the beam projecting on each side. The lengths of the projections are determined by the positions of the points of contraflexure in the beams, and at the end of each projection is formed the lower half of a scarfed joint similar to those of the roof member. The central portions of the beams are cast separately and have the upper halves of the scarfed joints at their ends.

Precast gutters, 1½ in. thick, in each valley of the roof and at the top of the external walls, support the lower end of the glazing and span between the roof members. The roof slabs are 2 ft. and 2 ft. 6 in. wide and have longitudinal ribs 7¼ in. deep at each edge with transverse ribs 3¾ in. deep at 2-ft. 6-in. centres spanning between them. Each rib is

Fig. 1.

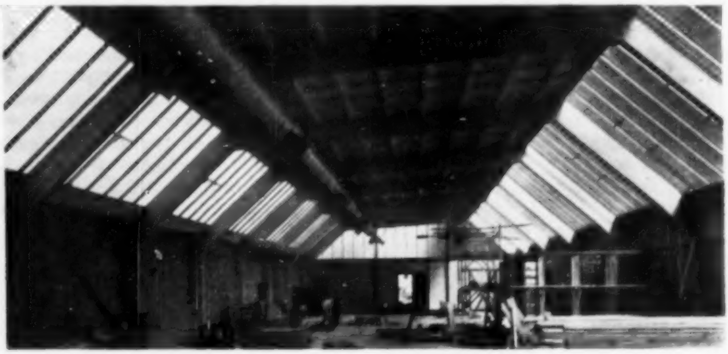


**Fig. 2.—Interior.**

reinforced, but the tops of the slabs, which are ¾ in. thick and coffered on the underside, are unreinforced. The bottoms of the slabs in the central bay are covered with acoustic boards which are placed in the bottom of the moulds before the concrete is cast and so form cavities between the ribs; to protect the boards their edges and undersides are temporarily wrapped in paper before they are placed in the mould. The slabs are bolted to the roof members and are covered with 2 in. of vermiculite and roofing felt with a mineral finish.

All the precast concrete was made at the site and comprised a 1 : 1½ : 3 mixture with a crushing strength of 4500 lb. per square inch at seven days.

The architect is Mr. Clive Pascall and the main contractors were Messrs. Y. J. Lovell & Son, Ltd. The reinforced concrete superstructure was designed and built by the London Ferro-Concrete Co., Ltd.

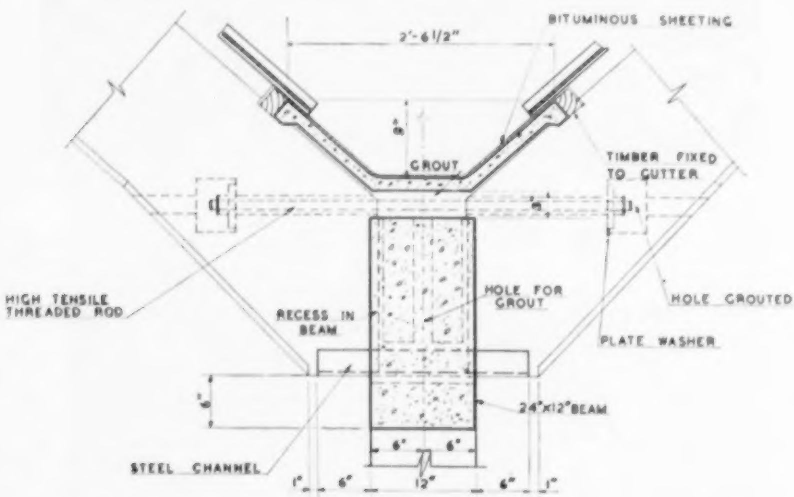



**Fig. 3.—Detail at Junction of Roof Members at Valleys between Bays.**

# Prestressed Bridge in Ireland.

THE original design for this bridge (*Fig.* 1) across the Kilkerrin estuary consisted of three 60-ft. arches but, owing to the difficulty of constructing foundations, a single-span bridge in prestressed concrete was proposed and was accepted by the consulting engineer, Mr. Nicholas O'Dwyer, of Dublin. The span between the centres of the bearings is 180 ft. The overall width is 26 ft., comprising a roadway of 18 ft. and two 4-ft. cantilevered footpaths. The end piers are 23 ft. high. The piers and the four main beams are prestressed, the cables being supported by light mild steel stirrups. The deck slab and footpaths are in reinforced concrete. The bridge was designed for the full British Ministry of Transport loads.

For the stagings, trees were obtained from a wood about 20 miles from the site, and the main props were set in concrete at low water. All the concrete was cast in position. The piers and beams were designed as a two-pinned prestressed frame. Since the piers are relatively very stiff, the horizontal thrust is appreciable and considerably affects the stresses at mid-span. The outward thrust at the base of the piers is restricted by the rock on which the piers are founded, and in the construction explosives were used to blast the rock.

The beams are parabolic, with deep haunches at the piers. This end stiffness effects an appreciable reduction in the bending moment at mid-span, where the overall depth is 3 ft. 3 in. only. The beams were prestressed by the Freyssinet post-tensioning system. Each cable comprises twelve 0·276-in. diameter wires tensioned by hydraulic jacks, and exerting a compressive force on the concrete of between 40 and 45 tons per cable. The cables are in 19-gauge seam-welded tubing. Altogether 162 cables were used with a total length of 11,600 ft. The maximum compressive stress in the concrete is about 2400 lb. per square inch in the bottom of the beams at midspan under dead-load conditions.

Fig. 1.

The concrete mixture in the central part of bridge was 1 : 1 : 2 and the test cubes had compressive strengths at 40 days varying from 8000 lb. to 10,500 lb. per square inch. In the rest of the work a 1 : 1½ : 3 mixture was used, the lowest compressive strength of which was 7000 lb. per square inch at 28 days. Internal vibrators were used to consolidate the concrete in the beams and piers.

The bridge and approach roads were built in rather less than six months and were completed in February 1954. The cost (including approach roads) was about £27,000. The hand-railing consists of galvanised tubular steel supported by standards of channel section which are grouted into pockets at the sides of the footpaths. The prestressed concrete

design was prepared by the British Reinforced Concrete Engineering Co., Ltd., who also supplied the prestressing cables and supervised the stressing operations at the site. The contractors were the Roe Quarry Co., Ltd., Dublin.

# Bridge Designed for Mining Subsidence.

SANKEY BROOK bridge (*Fig.* 1) is on the Burtonwood road diversion, near Warrington, Lancashire, and is designed to permit some settlement on very weak soil. This bridge, and a similar one over the river Winster at Meathop, was designed by Mr. James Drake, B.Sc., M.I.C.E., Surveyor and Bridgemaster of the Lancashire County Council. The Meathop bridge, with very similar ground conditions to those at Sankey Brook, has been in service for three years and shows no signs of movement.

The principle is to distribute the load by a rigid box design, and thereby keep the pressure on the ground to about half a ton per square foot. The wing-walls are cantilevered from the main structure, and vary in thickness from 1 ft. 6 in. at the bottom to 1 ft. at the top. As is seen in *Fig.* 2, the main reinforcement in the wing-walls is joined horizontally to the reinforcement in the main walls of the box, and economy is achieved by shaping the wing-walls to follow the ground line. The wing-walls have no footings and, should settlement or tipping of the main box occur, they should follow the settlement of the main structure, so that no cracking is likely to occur at the joint between the wing-walls and the main structure, and no such cracking has occurred.

The main box has a span of 25 ft. and a height between invert and roof of 15 ft. 6 in. and is designed for various combinations of load (for example, full active earth pressure on one wall with no pressure on the other wall and full live load on the deck). These various combinations of loading produce maximum positive and negative bending moments throughout the structure and provide for all possibilities of tipping and settlement.

When it was excavated the silty subsoil was so wet and unstable that the site of the box was completely enclosed in a cofferdam of steel sheet piles, which was incorporated in the base of the structure. Analysis of the river water indicated the presence of a harmful amount of sulphates, and the concrete in the invert and side walls was therefore made with sulphate-resisting cement.

Although no subsidence due to mining is anticipated at the site of the bridge there is a likelihood of it occurring farther upstream, and the Rivers Board requested that the invert be 4 ft. lower than the present level of the river-bed in order that the bed may be regraded if necessary. The contractors for the work were Messrs. Leonard Fairclough, Ltd., who also constructed the bridge at Meathop.

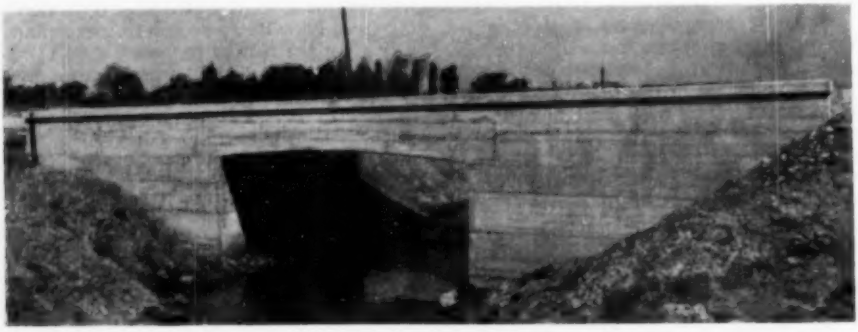


**Fig. 1.—Bridge at Sankey Brook, Lancashire.**

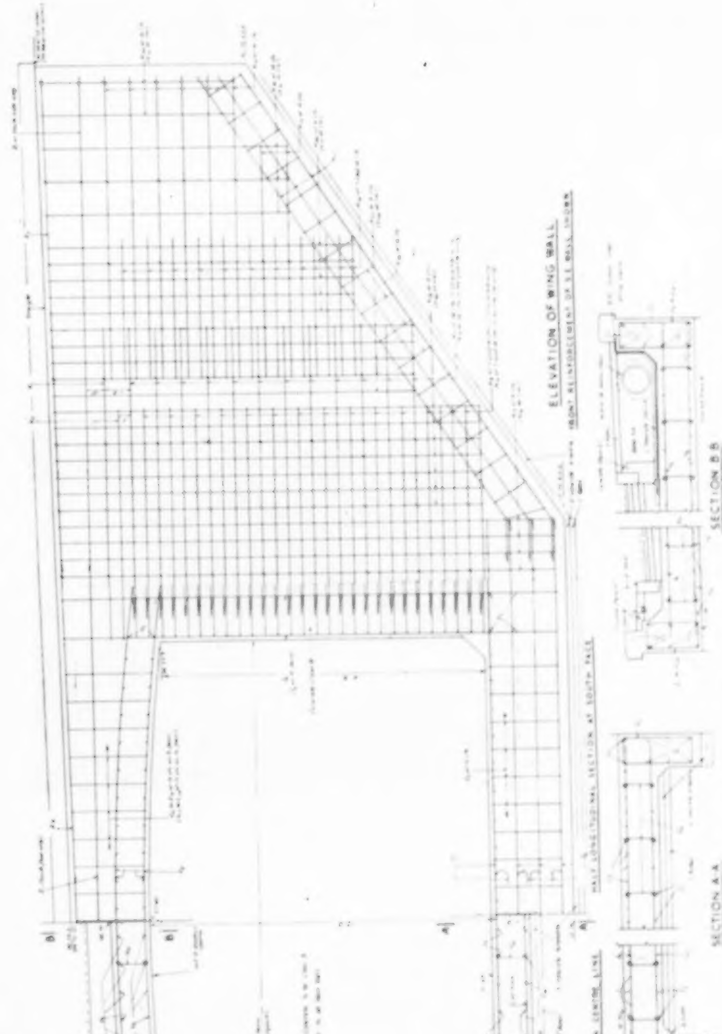



**Fig. 2.—Details of the Bridge.** (*See p. 40.*)

# A Prestressed Railway Bridge.

A BRIDGE (*Fig.* 1) to carry an additional track over the river Leen near Radford is part of the work of widening the line between Nottingham and Mansfield. The bridge has a skew span of 60 ft. and is of the through type (*Fig.* 2). The main girders are at 12 ft. centres and a floor 10 in. thick spans between them at the level of the bottom flanges. The main girders, of I section, are 6 ft. deep with flanges 2 ft. 3 in. wide and webs 9 in. thick. Each girder was prestressed with three Magnel-Blaton cables. Two cables in the bottom flange are straight, and particularly at each end of the bridge where, owing to the skew beam, the cables are close together. Internal vibrators were used throughout the work. Concreting was carried out in three stages, namely, half the length of the floor and the bottom flanges of the main girders, the remaining half of the floor, and the webs and top flanges of the main girders. Steel and timber shutters were used and they were supported on steel beams erected on the new abutments at each end and on temporary piles driven into the river-bed at mid-span.

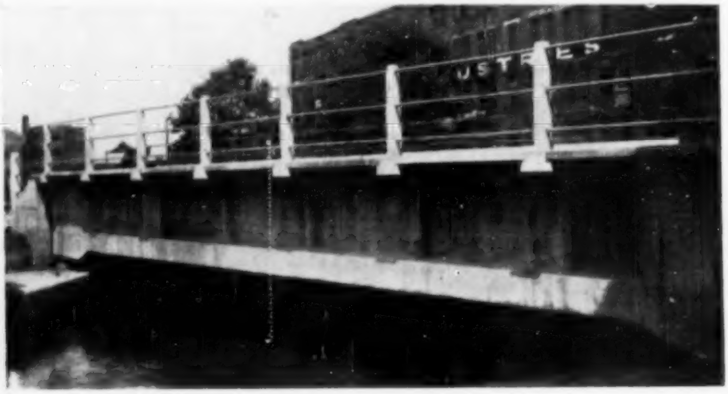


**Fig. 1.—The Completed Bridge.**

each comprises 40 wires of 0·276 in. diameter; the third cable is curved and comprises 32 similar wires. The floor is thickened at each end to form beams that span between the main girders at the bearings and are prestressed by ten cables. The floor is prestressed with Freyssinet cables, each composed of twelve wires of 0·2 in. diameter.

The bridge was concreted in situ, using ready-mixed concrete with a specified strength at 28 days of 7000 lb. per square inch and with the addition of 1 per cent. of a plasticiser to assist in obtaining a compact concrete, which was required

The two new abutments are of unreinforced concrete faced with bricks, and were constructed inside cofferdams of steel sheet piling. Rocker-bearings of meehanite cast iron are provided at each end of the main girders, and these were set in position before the steel beam for supporting the shuttering was erected.

The work at the site was carried out by direct labour. The design was prepared at the Regional Headquarters under the direction of Mr. J. Taylor Thompson, M.C., M.I.C.E., Civil Engineer, British Railways (London Midland Region).

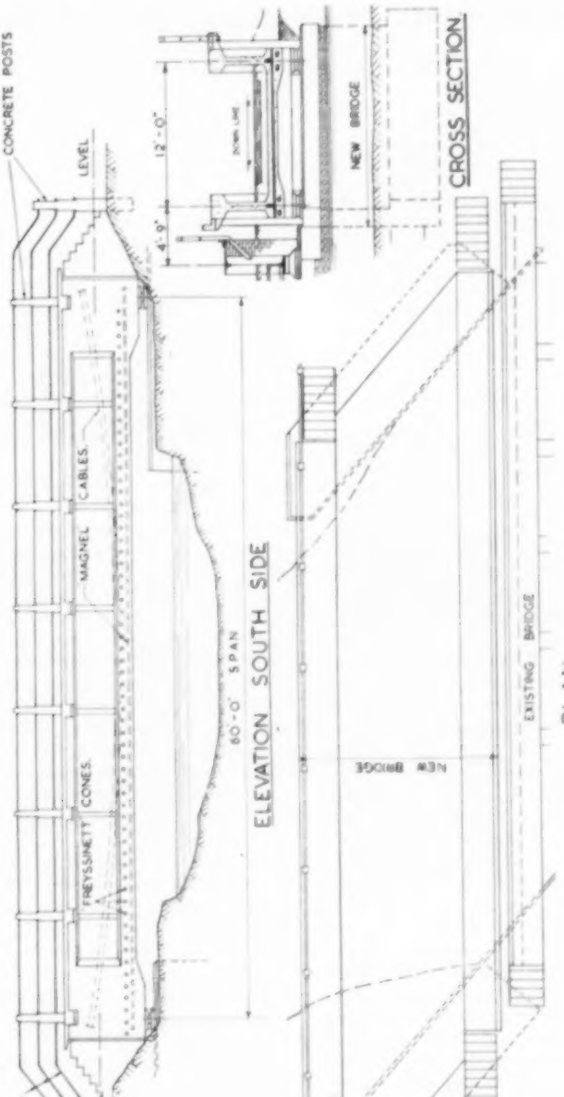



**Fig. 2.—Elevation and Plan.** *(See p. 42.)*

# Residential Flats at Wandsworth, London.

**RAPID CONSTRUCTION.**

THE housing scheme at Trinity Road, Wandsworth, for the London County Council consists of five eleven-story blocks of flats each with two-story pent-houses, a separate boiler house and a club room. The blocks measure about 65 ft. by 61 ft. 6 in. on plan and have four flats per floor, two staircases, and two lifts, each lift serving alternate floors. The structures comprise 7-in. reinforced concrete floor slabs, precast concrete staircases, 6-in. internal load-bearing reinforced concrete walls, external reinforced concrete columns, and reinforced concrete edge-beams.

Blocks Nos. 1 to 3 have raft foundations and blocks Nos. 4 and 5 have piled foundations. The rafts consist of bases 2 ft. 6 in. deep bearing on sandy gravel and connected by a 9-in. reinforced concrete slab. The in-situ piles were designed for 40 tons working load and were made with sulphate-resisting cement; they have a nominal diameter of 17 in. and mean tube lengths of 27 ft. 9 in. under block No. 4 and 21 ft. under block No. 5. Tests made on one pile for each block showed a total recovery in the case of block No. 4 and a permanent settlement of $\frac{1}{16}$ in. in the case of block No. 5 when the load of 75 tons had been removed.

Prior to starting the work discussions were held to produce a scheme that would make full use of the mobile tower crane which the contractor proposed to employ. This crane has a jib of 65 ft. and a maximum operating height of 330 ft.; it is capable of lifting $17\frac{1}{2}$ cwt. at its full extent, increasing to 3 tons at 15 ft. radius. It was decided that, in addition to the staircases, the edge beams, lintels, and balconies should be precast; an added advantage was that the balconies and staircases were to be fair-face work, and a much better finish was obtained by precasting. It was also decided not to use scaffolding, and that when the structural work was completed the exterior brickwork, facing, etc., should be erected by using a hanging scaffold and working from cradles. Shuttering was resin-bonded plywood in complete wall lengths, stiffened by 3-in. by 2-in. and 4-in. by 3-in. timber.

The sequence of constructing a floor was an important factor, and it was necessary to make use of the time generally wasted while concrete matures. Work was confined to one block at a time, thereby avoiding waste of men's time that may arise when work is spread over a large site, and closer supervision was also possible.

The walls were concreted the full story-height of 8 ft. $2\frac{1}{2}$ in. at one operation, and the concrete was placed directly by the crane, moving a skip along the wall to maintain a level top surface and thoroughly tamping with long rods. The top of the concrete already placed was grouted before the next story-height was concreted. The reinforcement was partially prefabricated in welded "ladders," and precast concrete spacers ensured that the required cover was maintained. While the wall shuttering was still in position the bricks and blocks necessary for the external cladding and internal partitions were hoisted by the crane and placed on the floor slab. Twenty-four hours after concreting the wall shutters were removed to be cleaned ready for the next lift. Props were used to support at midspan the telescopic centering used for three floors that spanned 18 ft. between edge beams or walls and the beams that rested in pockets in the main walls.

The precast balconies were erected at the same time as plywood shuttering for the floors, and the slab was then concreted directly from the crane-skip without the use of barrows. Lastly the precast staircase was erected. During the wall construction, precast concrete nibs were fixed into the wall shutters and on these the landings were bedded, the flights spanning between the landings.

To maintain full employment of all trades each block was erected in stages, one half being nearly a floor ahead of the other. By using these methods a story was built in as short a time as five working days, and the average was less than seven days per story throughout the block. *Figs.* 1 to 3 show the progress of the work.

The pent-house containing the water

Fig. 1.—Ten Weeks after Starting Excavation for Foundation.

tanks, lift, and ventilating machinery is suspended from two frames and walls to avoid a heavy load coming directly on to the roof slab. The boiler house is a separate structure, but the vertical flue is contained within the walls of one of the blocks of flats, the underground connecting flue having an expansion joint at the junction with the boiler house.

Fig. 2.—Two Weeks Later.

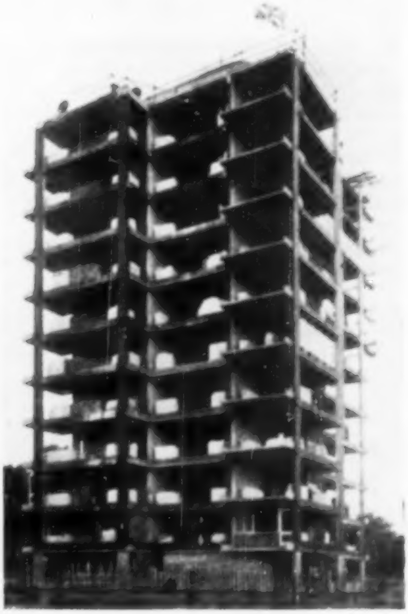


Fig. 3.—Eighteen Weeks after Starting Excavation for Foundation.

Dr. J. L. Martin, M.A., F.R.I.B.A., is the Architect of the London County Council, and Mr. T. Whitfield-Lewis the Principal Housing Architect. Messrs. Ove Arup & Partners are the consulting engineers and Messrs. Wates, Ltd., the contractors.

### Testing the Consistency of Concrete.

It is reported that many public authorities in U.S.A. (particularly road authorities) are using a method of gauging the consistency of fresh concrete developed at the Engineering Materials Laboratory of the University of California. The test is based on the penetration into the concrete of a ball of 6 in. diameter and weighing 30 lb.

# A Waste-paper Store.

A WASTE-PAPER store at Merton Abbey, Surrey, for the New Merton Board Mills, Ltd., consists of two bays each 44 ft. wide by about 300 ft. long by 46 ft. 6 in. to the eaves. One bay is provided with a 2-tons and the other with a 4-tons overhead travelling electric crane. The east end of the building (*Fig.* 1) is of reinforced concrete up to the first floor, which is designed to carry a superimposed load of 7 cwt. per square foot. The first floor supports steel stanchions which carry the gantry girders and pitched roof.

This portion of the building extends over the river Wandle for a length of about 100 ft., and it was first necessary for the river to be culverted. Owing to the poor bearing capacity of the ground, bored piles were used for all the foundations and also to support the new river walls. The culvert (*Fig.* 2) comprises reinforced concrete walls 2 ft. thick along

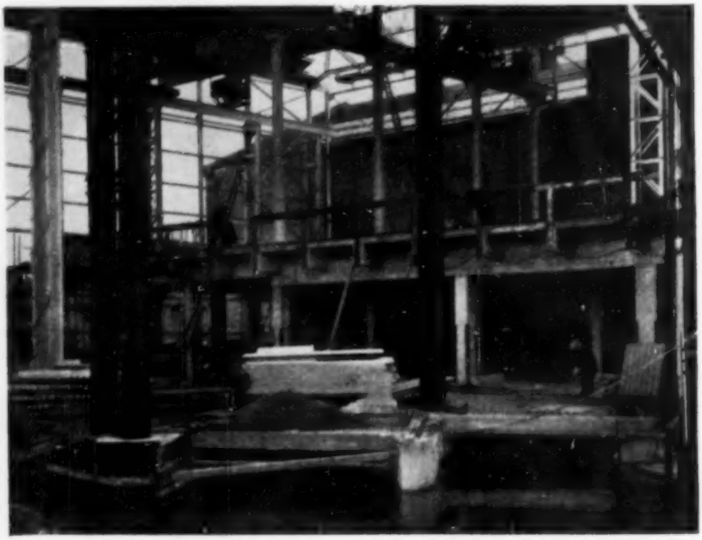


**Fig. 1.—Construction over the Culvert.**

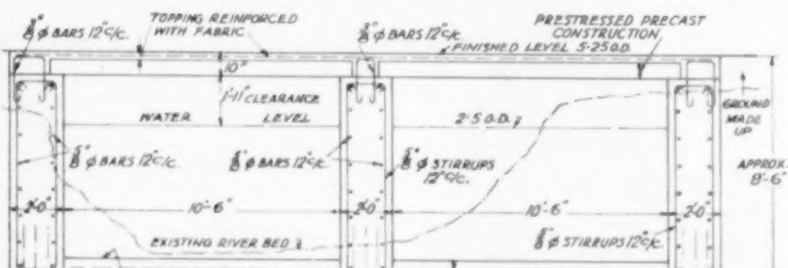



**Fig. 2.—Section through the Culvert.**

each bank and also along the centre-line of the river. These walls carry some of the reinforced concrete columns, 18 in. sq., supporting the first floor, and also the ground floor over the river. The ground floor is designed to carry a superimposed load of 4 cwt. per square foot and is of composite prestressed precast and in-situ concrete construction.

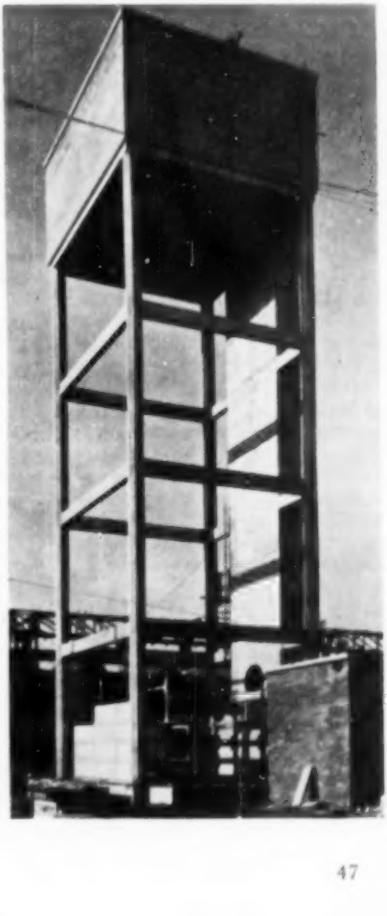
The culvert was constructed in two stages. First, sheet steel piling was driven to enclose one half of the river, thus enabling the piles to be placed and this side and the central wall to be constructed; the piling was then withdrawn and driven along the other bank to enable this wall to be built. The flow of water through the restricted width of the river during these operations was controlled by means of mill penstocks situated at the down-stream end of the new construction. The first floor is carried over the penstocks on a reinforced concrete beam 5 ft. by 2 ft. in cross section and with a span of 39 ft. The other floor beams are 3 ft. by 1 ft. 6 in. in cross section and span 14 ft. 8 in.

The consulting engineers were Messrs. John F. Farquharson & Partners and the contractors were the Demolition & Construction Co., Ltd. The bored piles were by the Pressure Piling Co., Ltd.

## Rapid Construction of a Water Tower.

THE water tower illustrated is 75 ft. high and has a capacity of 16,000 gallons. The structure was erected from ground level in six weeks.

Under the tower is a cooling pond in two compartments each about 18 ft. square by 16 ft. 9 in. deep, and a receiving compartment. The reinforced concrete floor slab is 2 ft. thick and the walls 9 in. thick. Above ground level the tank is supported on four octagonal reinforced concrete columns 1 ft. 6 in. across the flats and at 21-ft. 6-in. by 20-ft. 6-in. centres on plan. The horizontal braces are 8 in. wide by 1 ft. 6 in. deep and are at about 17-ft. centres vertically. The main pipe-duct is U-shaped on plan, and provision has been made for the pipes to be totally enclosed with precast slabs rag-bolted to the vertical faces. The tank has a beam-and-slab floor, walls $7\frac{1}{2}$ in. thick, and a roof 6 in. thick. The roof is surmounted by a parapet wall 3 ft. 6 in. high finished with a precast concrete coping.



Tubular-steel scaffolding was used, both for access and for aligning and supporting the shuttering. Steel shuttering was used throughout. The materials for the concrete were measured by weight; the mixer discharged into a hopper which was hoisted to the required level and the concrete was distributed by barrows. All the concrete was consolidated by internal and external pneumatic vibrators. The consultants were Messrs. Henry M. Hale & Partners, and the contractors were Messrs. Robert M. Douglas (Contractors), Ltd.

# A Prestressed Water Tower.

WHAT is believed to be the first prestressed concrete water tower in Great Britain was completed at Meare for the Wells Rural District Council in 1953 as part of a scheme including four circular prestressed concrete reservoirs one of which has been constructed. The tank is 37 ft. 5 in. diameter by 23 ft. 9 in. deep and has a capacity of 125,000 gallons. Top water-level is 80 ft. above ground level. The tank (*Fig.* 1) is designed so that the walls will always be in compression.

The tank is carried on eight reinforced concrete columns and a central shaft. The pumphouse and suction tank, which are partly below ground level, are in the base. The pipework and access ladders are in the central shaft, which provides access to the roof. The floor consists of eight conical sections with their apices at the central shaft. Radial edge-beams are placed between them, and the floor is stiffened by a reinforced concrete ring-beam. The radial beams, the ring-beam, and the whole of the floor were cast in one operation without construction joints. The part of the central shaft that forms the inner wall of the tank is prestressed vertically by four 12-wire cables at quarter-points. The roof is a reinforced concrete slab 6 in. thick.

The outer wall is 6 in. thick and is prestressed vertically and circumferentially by the Freyssinet system. The vertical prestressing is by 48 cables equally spaced and each containing twelve wires of 0·2 in. diameter. The cables are in steel sheaths in the centre of the wall; they are anchored in ring-beams at the base and at the top. The circumferential prestressing is applied by 2-wire cables in grooves on the outside of the wall, 2½ in. apart at the base increasing to 3 in. at the top. Each cable passes half-way around the tank and is anchored in buttresses at the quarter-points, pairs of cables being anchored at alternate pairs of buttresses so that half the cables are anchored at each pair of opposing buttresses and half at the other. In addition to the structural buttresses, there are four extra buttresses at alternate quarter points for reasons of appearance.

The vertical cables were tensioned first, commencing with those diametrically opposed to one another, and proceeding in a staggered arrangement. The circumferential cables were then tensioned starting at the bottom. Due to friction between the cables and the grooves it was found desirable to ensure that the surface of the grooves was as regular and smooth as possible and to tension the cables in stages until the required extension was obtained. The frictional loss recorded was very close to the calculated loss. The cables were covered with ½-in. of gunite. The tank was tested before the gunite was applied and was found to be watertight. The gunite was

**Fig. 1.**

applied after the tank had been filled in order to reduce the risk of crazing.

The consulting engineers are Messrs. Sandford Fawcett & Partners and the prestressed concrete was designed in collaboration with Dr. T. O. Lazarides. The main contractors were the Vibrated Concrete Construction Co., Ltd.

# Residential Flats, Putney, London.

THE Ashburton Estate, Putney Heath, now nearing completion, is one of the larger housing schemes of the London County Council, and will provide 1251 dwellings. The buildings (*Fig.* 3) generally have brick walls, with floors, beams, and staircases in reinforced concrete. Load-bearing walls are used longitudinally in some of the blocks and transversely in others.

In the case of the transverse load-bearing walls with edge-beams carrying

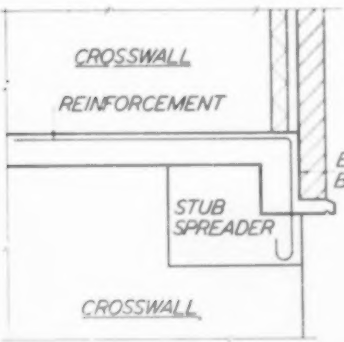



**Fig. 1.—Method of Distributing Load from Edge-beam.**

**Fig. 2.—A Detail of an Elevation.**

**Fig. 3.—Flats During Construction.**

brick facing panels, it has been general practice to construct a beam over the length of the wall to distribute the load from the edge-beams evenly across the wall. It was desired, however, to find an efficient yet more economical arrangement. After discussion the authorities agreed that, due to the bond of the brickwork, the load from an edge-beam would spread but the immediate pressure would be excessive without special provision, and as a result " stub spreaders " (*Fig.* 1) with tie-bars were introduced to distribute the load over sufficient length of the cross-wall so that the compressive stress in the brickwork does not exceed that allowed by L.C.C. By-law 5.22.

Two of the blocks have special features with shops, a connecting bridge, suspended pavements, and external staircases, and many blocks have external staircases enclosed in glazing (*Fig.* 2). The work was carried out under the direction of the London County Council Architect's Department. The contractors are Messrs. Rush & Tompkins, Ltd. The design and reinforcement for the reinforced concrete were supplied by the British Reinforced Concrete Engineering Co., Ltd.

# Workshop at a Technical College.

THIS workshop is part of a new technical college being built for the Kingston-upon-Hull Corporation. The building straddles one end of the site of a dock built about 1780 and filled in about twenty years ago to form Queen's garden. It is a single-story structure 312 ft. long by 192 ft. wide, planned on a grid of 24 ft. by 24 ft., and divided into four parts by transverse corridors 12 ft. wide. The frame (*Fig.* 1) is a series of north-light bays of reinforced concrete precast members prestressed after erection by the Lee-McCall system. The shell roof is also of precast units, and each north-light and roof unit together span 24 ft. The corridors have flat roofs of reinforced precast slabs resting on tubular rollers to allow for expansion and

**Fig. 1.—Erecting a Roof Slab.**

contraction. An entrance wing and temporary main corridor at present take the place of the entrance hall, which will later form part of the ground floor of the nine-story main block.

**Foundations.**—The soil above the boulder clay is poor, especially the filling within the dock area, and piled foundations were therefore used. Large foundation-beams span the dock walls throughout and provide a standard grid for the suspended ground floor, thus assisting in standardising the design. Because of the curve of the dock wall across the site and the nearness of the lock gate and its abutments, the foundations are independent of the wall because the latter's foundation is above the boulder clay on which the building is founded.

The suspended reinforced concrete ground floor contains ducts throughout its area for main services and for as many subsidiary services as could be concealed in this fashion; these ducts form the main beams of the floor and incorporate sockets for the fixed-ended columns. In the heat-engines laboratory parts of the floor are lower to permit machinery bases to be isolated against the transmission of vibration. The floor of the hydraulics laboratory incorporates a flume for experimental work.

**The Superstructure.** — Full-scale loading tests were made on the precast frames and the shell roof units at the beginning of the work. There are eight frames in each bay, and each bay comprises two basic units, namely a combined column and north-light strut, and a curved rib. The units, which are cast with holes for the prestressing bars, are erected with a derrick crane. The high-tensile bars are then inserted, the joints concreted, and the bars tensioned and grouted.

The gutter units are precast on the site. The prestressing bars are tensioned sufficiently to withstand handling stresses, and fully tensioned when the gutters are in position. These gutter units, when connected to the main frames, make a series of portal frames laterally.

Each unit of the shell roof is 2½ in. thick, measures about 16 ft. by 24 ft., spans in the 24-ft. direction, and weighs about 7 tons. These units are all precast and prestressed on the site, in batches of eight, on a bay of the previously-constructed ground floor. Eight gutter units are cast simultaneously with eight roof units. The roof units are placed on the frames, clamped down, and in-situ concrete placed between adjacent units over the ribs of the frame.

**Prestressing.**—The prestressing bed (*Figs.* 2 and 3) is 200 ft. long and the jacking head, which can be used over its full width, is about 20 ft. long; the total compressive force that can be applied is about 450 tons. To avoid bending moments on the floor, rocker bearings are provided which transmit the load axially. A rolled steel joist cast in the floor acts as a spreader and is anchored down to provide the necessary frictional resistance to any vertical movement of the upright channels. The forces exerted by the prestressing wires, which are about 15 in. above the floor slab, are counterbalanced by high-tensile steel tie-bars buried in drain pipes in the ground 5 ft. below the floor; the pipes permit freedom of movement of the tie-bars, which can be reclaimed on completion. The wires are tensioned in pairs by the travelling jack, which has a compensating device in the gripping head; the load is simultaneously taken up on the high-tensile steel bars with a Lee-McCall jack in order to ensure that the channel uprights remain vertical.

To release the pull the loads on the tie-bars are gradually released and the entire head arrangement is allowed to pivot about the rocker bearings until all the load is released. To reduce the risk of damage to the threaded ends of the tie-bars, knife-edge bearings are provided behind the nuts.

The timber moulds rest on steel scaffold tubes bedded in mortar. This facilitates levelling, and also enables the moulds to slide freely when the pull is released. The ends of the moulds for the roof units are steel plates drilled for the 193 0·2-in. diameter wires. The formers, made of rolled steel joists, are similarly drilled; these span the full width of the bed in order to prevent the vertical force from the sloping wires transmitting bending moments to the floor.

The concrete mixture is 1 : 1 : 2 with a water-cement ratio of 0·34. Washed Trent river sand and gravel and rapid-hardening Portland cement are used. A compressive strength of 5000 lb. per square inch is obtained in two days, when the pull is released. The concrete is placed by crane from bottom-opening skips and compacted with a pneumatically-

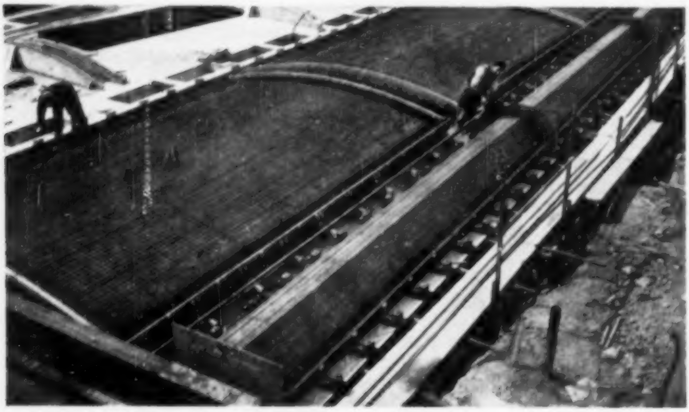


**Fig. 2.—Precasting Roof Slabs.**

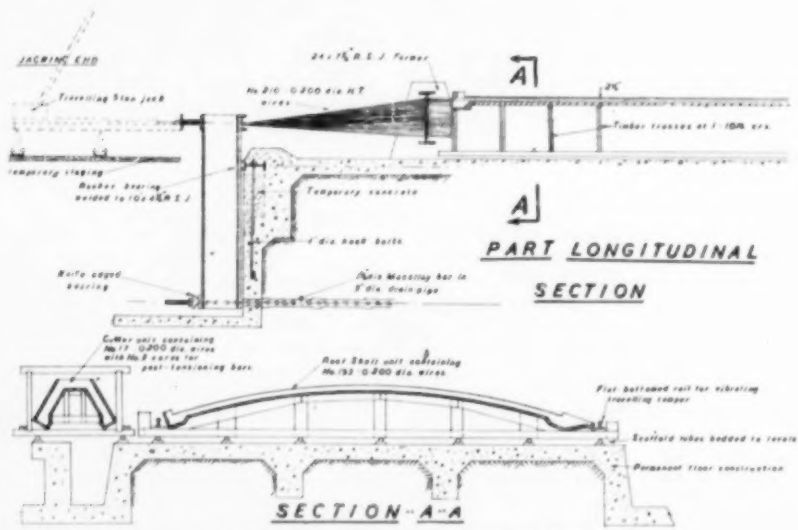



**Fig. 3.—Details of the Casting Bed.**

vibrated travelling tamper shaped to the contour of the units.

A crane is used to transport the units, and they are erected by another crane. The units are lifted in a frame which consists of two steel channels connected by welded steel tubes and a sling (*Fig.* 1). This frame is fixed to bolts cast in the units and later burnt off.

The architect is Mr. F. Gibberd, C.B.E., F.R.I.B.A., and the consulting engineers Messrs. Scott & Wilson, Kirkpatrick & Partners. The general contractors are Messrs. William Moss & Sons, Ltd. The piling was carried out by Holmpress Piles, Ltd., and the precast frames were made by Trent Concrete, Ltd.; the prestressing bars were supplied by McCalls Macalloy, Ltd., and the prestressing wires by Messrs. Richard Johnson & Nephew, Ltd.

# Residential Flats and School, Paddington, London.

THE Hallfield estate, Paddington, consists of fourteen blocks of flats and a school for 720 children. The estate is being developed in two stages; the first stage consists of two ten-story blocks (80 flats each) and four six-story blocks of 22 flats each making a total of 248 dwellings, and the second stage comprises four ten-story blocks and four six-story blocks comprising 408 dwellings. Stage one is completed; of stage two about three-quarters of the structural work is completed. Underground calorifier rooms for each stage provide heating and hot water; the school is also heated by the calorifier supplying the second stage of the work. An extension to the boiler house at the public baths (400 yd. from the site) supplies the steam for heating. All the pipework is in concrete ducts with removable precast covers.

### The Flats.

There are 238 piles under each ten-story block and 77 piles under each six-story block. All the piles have a carrying capacity of 40 tons, and sulphate-resisting cement was used throughout. Both precast and in-situ piles were used.

The ten-story blocks, with reinforced concrete load-bearing cross-walls, produced a high proportion of repetitive construction. The reinforced concrete cross-walls are at 23 ft. centres; they are 8 in. thick on the ground floor and are reduced in stages to 5 in. thick at the seventh floor. The end of a ten-story block is seen in the background of *Fig.* 1. The stresses in the walls are generally low, and the reinforcement is generally 0·2 per cent. vertically and 0·1 per cent. horizontally of the gross cross-sectional areas of the walls.

The full story-height of the cross-walls was concreted in one operation. The floor slabs have 23 lb. per square yard of cold-worked square twisted bars; the design stresses were 27,000 lb. per square inch in the reinforcement and 1000 lb. per square inch in the concrete. A 1:2:4 mixture was used throughout.

The cantilevered access galleries are supported by a beam formed in the depth of the 8-in. floor slab plus the 5-in. thickness of the gallery (13 in. overall) by 5⅞ in. wide. This was possible by having twin door-columns at the entrance to each flat, and so reducing the regular 23-ft.

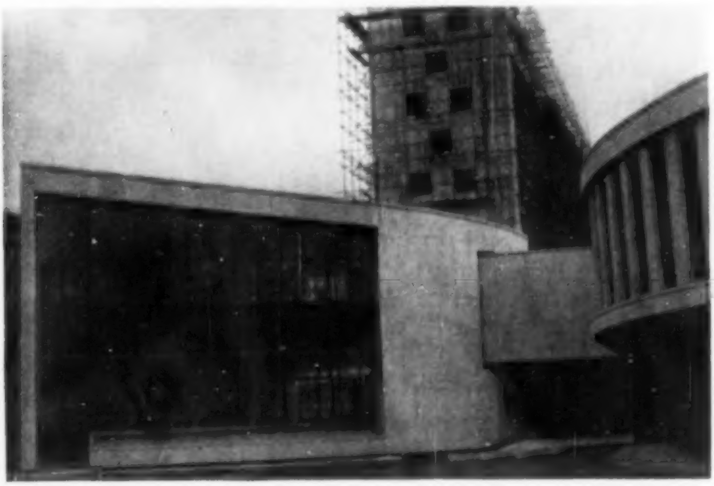


**Fig. 1.—The Assembly Hall. Part of Ten-story Block in Background.**

spans between the cross-walls. These door columns, finished in fair-faced concrete, are used as an architectural feature. The relieving bending moment due to the cantilevered access gallery made it possible to reduce the longitudinal reinforcement in the 8-in. floors up to a point about 10 ft. from the gallery beam. By dividing the structure, which is 230 ft. long, into three parts, two of the cross-walls at the ninth-floor level were made as two separate 4-in. walls with insulating board 1 in. thick between them ; these provided expansion joints at roof level. From ground to first-floor level the end walls are supported on reinforced concrete columns (*Fig.* 2).

The six-story blocks are similar in construction to the larger blocks. The access gallery is supported in a similar manner to that used in the ten-story blocks. The six-story blocks are 115 ft. long, and expansion joints were not considered necessary. Precast concrete facing slabs are used as a finish, particularly on the end walls of the six-story blocks. These slabs, $1\frac{3}{4}$ in. thick, have a Portland stone surface finish and are reinforced with $\frac{3}{16}$-in. diameter bars. Two of these bars project from the backs of the slabs and are embedded in the concrete walls. The precast slabs were used as permanent

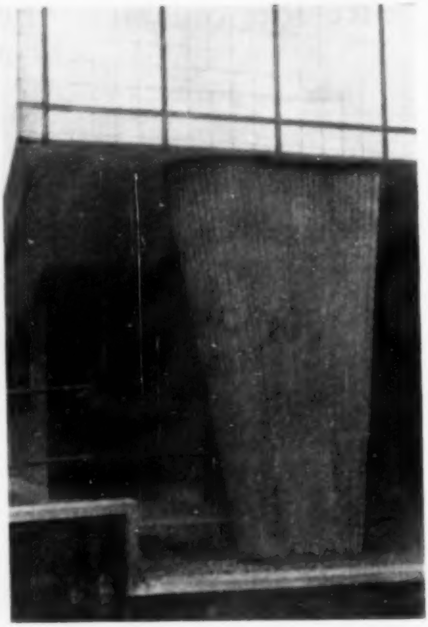


**Fig. 2.—Columns Under Corner of Ten-story Block.**

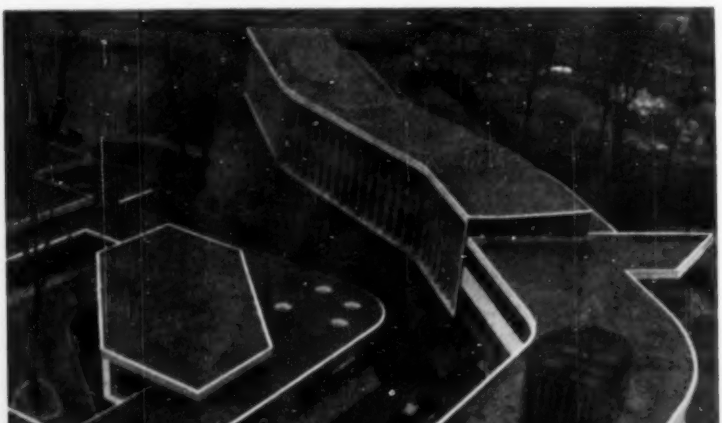


**Fig. 3.—General View of School Buildings.**

shuttering, supported during concreting by vertical timbers bolted through the inner shutters. Half-round holes in adjacent slabs allowed the bolts to pass through; these holes and all the joints were subsequently filled with a sealing compound. Precast slabs were used also on two of the ten-story blocks; the other four ten-story blocks having a tiled finish.

### The School.

The school (*Fig.* 3) comprises a single-story structure for infants and a two-story structure for juniors and administration. The infants' classrooms are in pairs; the dividing brick wall and tubular-steel columns, one in each corner of the classroom, support the roof. The roof is 10 in. thick overall, and comprises hollow tiles between ribs 3 in. wide. The steel columns are 6 in. diameter with plates 12 in. square by ½ in. thick welded to the top and bottom.

In the two-story block the floors are two-way in-situ concrete slabs 8 in. thick supported on 9-in. brick walls between classrooms and by a beam and column between the classrooms and the corridor on the north side. Support on the south side is provided by precast mullions at 4 ft. centres. These mullions were precast in one piece, with projecting reinforcement to bond into the in-situ slabs at first-floor level and roof level. A standard type of excavator was adapted to lift the mullions and lower them into precast stools that had been grouted into pockets left in the foundation, a projection on the mullion being grouted into a pocket in the stool. A similar arrangement of mullions is used on the south elevation of the dining hall.

On the north side of the administration building precast concrete louvres 3 in. thick support the roof over the main corridor. The whole of the corridor is

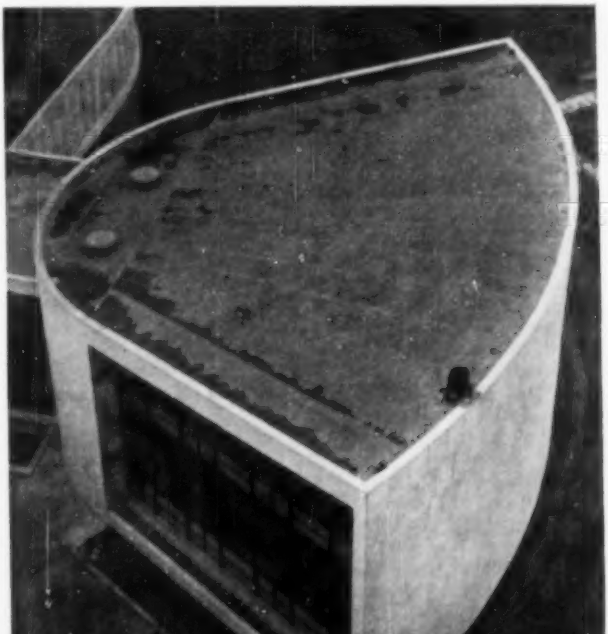


**Fig. 4.—The Assembly Hall.**

cantilevered from the curved wall at first-floor level.

The two-story assembly hall (*Fig.* 4) is supported by reinforced concrete columns at the south end and on a curved concrete wall 5 in. thick at the north end. The first floor is of hollow tile construction 1 ft. 6 in. thick. The maximum span in the assembly hall is 39 ft., and the hollow tiles are arranged radially on the lines of the lighting system, tiles being omitted at points where inset lighting units were required. A similar type of construction 15 in. deep was used at roof level. The roof is insulated by foamed-slag concrete weighing 65 lb. per cubic foot of an average thickness of 4 in. Precast facing slabs were used as permanent shuttering to the external walls.

The main contractors for the first stage of the housing scheme were Messrs. Walter Lawrence & Son, Ltd., and for the second stage Messrs. F. G. Minter, Ltd. Messrs. Allen, Fairhead & Sons, Ltd., were the contractors for the school, and Messrs. Wates, Ltd., made the precast work. The architects for the housing scheme are Tecton, and the executive architects Messrs. Drake & Lasdun, who are also architects for the school. Messrs. Ove Arup & Partners are the consulting structural engineers for both schemes.

## Lectures on Building.

THE following lectures have been arranged by the Ministry of Works. Admission is free.

Structural Problems of Multi-story Housing, by W. A. Fairhurst. Institution of Engineers and Shipbuilders, 39 Elmbank Crescent, Glasgow, C.2. January 18. 7.15 p.m.

Problems of Plastering and Rendering, by L. A. Ragsdale. 2 Pickford Street, Aldershot. January 18. 7 p.m.

Thermal Insulation of Building Structures, by J. S. Alton. Technical College, Bell Street, Wakefield. January 18. 7.15 p.m. By A. H. Palmer. College of Art and Technology, The Newarke, Leicester. January 26. 7.15 p.m. By J. Lawrie. Technical College, Broadway, Dudley. January 26. 7.15 p.m.

Trends in the Development of Mechanical Plant for the Building Industry, by W. R. Matthews. Technical College, Lichfield Road, Great Yarmouth. January 19. 7.30 p.m.

Alternative Forms of Construction, by L. R. Creasy. College of Technology, Warren Street, Sheffield. January 20. 7.15 p.m.

Discussion : That Greater Use of Alternative Materials would give Increased Productivity in the Building Industry. Public Library, Scarborough. January 20. 7 p.m.

Some Aspects of the Building Contract, by Norman P. Greig. Portland Hall, Little Tichfield Street, London, W.1. January 20. 7 p.m.

Field Maintenance of Builders' Plant, by J. Stafford. Technical College, St. Mary Street, Southampton. January 25. 7 p.m. At Technical College, Sunderland. January 31. 7 p.m.

Discussion : The Importance of Technical Education in the Building Industry. Technical College, Northgate, Darlington. January 25. 7 p.m.

Problems of Plastering and Rendering, by E. L. Westbrook. Bodhyfryd Hall, Chester Road, Wrexham. January 25. 7 p.m. Council Offices, Rhyl. January 26. 7 p.m.

Prestressed Concrete, by J. S. Arlett. Technical Institute, Stockton-on-Tees. January 26. 7 p.m.

Shell Roofs, by R. Jones. Technical College, Queen Street South, Huddersfield. January 26. 7.15 p.m.

Legal Obligations of Building Contractors, by John J. Clarke. Technical College, Hapwood Lane, Halifax. January 27. 7.15 p.m.

Essentials of Good Concreting, by E. E. H. Bate, O.B.E., M.C. Technical College, School Street, Walsall. January 27. 7.15 p.m.

# A Distillery in London.

Plain concrete, reinforced concrete, precast concrete, and both pre-tensioned and post-tensioned prestressed concrete are used in the construction of a gin distillery on a site of two acres off Goswell Road, London. The work is in three main stages. The first stage, comprising the stillhouse, warehouse, and boiler house, is completed; the second stage, comprising the bottling and case departments, is in course of construction; the third stage (the main offices) is about to be started. The following notes deal with the first stage, but the principles of design and construction apply also to the other parts of the work.

Due to the very heavy floor load of 4½ cwt. per square foot, plain concrete foundations have been taken down to ballast some 25 ft. below road level; this gave rise to interesting problems of shoring and underpinning adjoining properties and roads.

The main hall of the stillhouse is 100 ft. long, 54 ft. wide, and 78 ft. high, and is spanned at roof level by nine prestressed concrete beams (*Fig.* 1) supported on the reinforced concrete structure. The beams, which weigh 6½ tons each, are I section with a 5-in. web, increasing in depth from 2 ft. 5 in. at the ends to 3 ft. 3 in. at mid-span. Each beam consists of five precast sections assembled with dry-packed mortar joints and prestressed on the second floor of the stillhouse. The ducts for the five 12-wire Freyssinet cables were formed in the precast segments with well-greased 1½-in. rods which were removed immediately the initial set of the concrete had taken place. An erection mast 80 ft. high was used to hoist the beams from the second floor to the roof. Precast concrete units span between the main beams. A reinforced concrete stairway is supported on half-landing slabs cantilevered from the wall beams between the floors. *Fig.* 2 shows the stillhouse nearing completion.

The four-story duty-paid warehouse, also designed for a floor load of 4½ cwt.

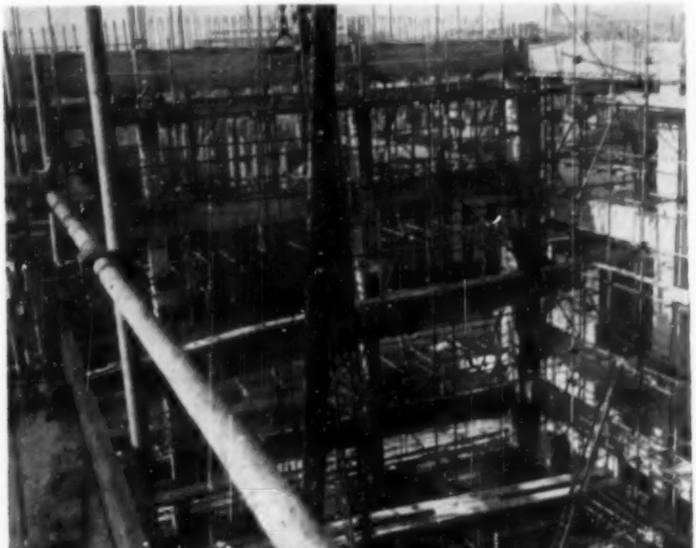


**Fig. 1.—The Stillhouse during Construction.**

per square foot, is a framed structure and contains an asphalt-lined reinforced concrete water tank of 500,000 gallons capacity at main roof level. A beam 8 ft. deep at first-floor level spans the loading dock opening, which is the full width of the warehouse, and carries concentrated loads from the columns on the upper floors.

The roof-beams of the boiler house (*Fig.* 3) were pre-tensioned on the long-line system and delivered to the site ready for erection; they span 40 ft. and carry

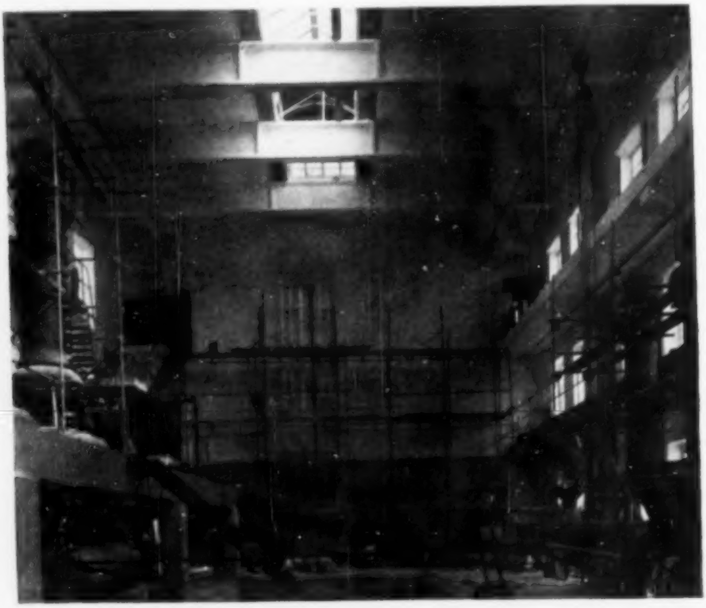


**Fig. 2.—Stillhouse nearing Completion.**

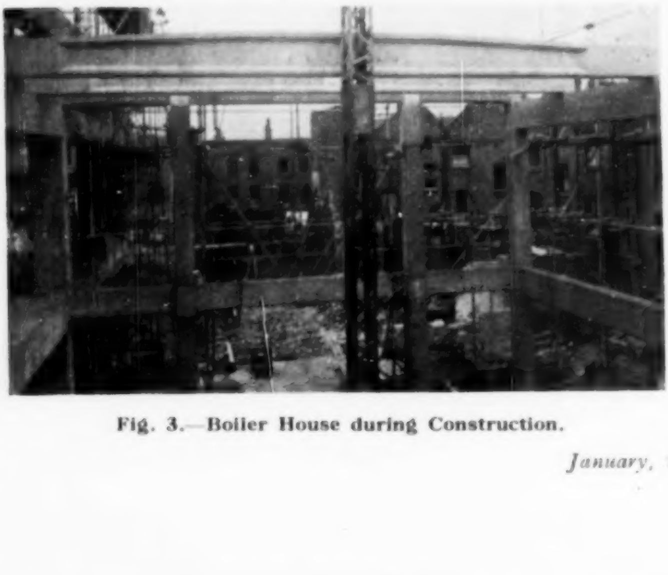


**Fig. 3.—Boiler House during Construction.**

precast concrete roof units. There are two oil-fired boilers and a circular brick chimney 100 ft. high.

Strict fire precautions controlled the design of the structure and 2½ in. of cover were required in addition to wire mesh to give a fire-safety period of four hours.

The elevation comprises 2-in. facing bricks of bronze hue, Portland stone dressings, and polished Cornish granite facings to the loading dock and pavement plinths.

Messrs. Chamberlain & Willows, building surveyors, who were appointed by Messrs. Tanqueray Gordon & Co., Ltd., to develop the site, appointed Messrs. Stroyer & Adcock to be responsible for the structural design, and the construction and auxiliary works are by Messrs. Holland & Hannen and Cubitts, Ltd.

# Concrete Roads in Germany.

THE following notes are abstracted from a paper by Dr. E. H. Graf in the American journal "Rock Products."

The cement should be of good quality with a high content of $Fe_2O_3$ and moderate contents of $Al_2O_3$ and MgO. The fineness should be such that there is a residue of at least 5 per cent. on a sieve with 176 meshes per square inch. The initial set should occur in less than one hour at an air temperature of 95 deg. F. The cement should have a high strength in bending and little shrinkage on hardening. The cement should not be delivered until after its strength at seven days is known in order that it shall not be used as soon as it is made; shrinkage cracks have occurred as a result of using fresh cement in hot weather.

Cement and sand should be measured by weight. Coarse aggregate may be measured by volume. The concrete should be consolidated to as great a density as possible. Most of the variation in the strength of the concrete, as shown by tests of cores, is due to variable consolidation. In roads laid in two courses the concrete in the two layers should have the same strength and elasticity, and the top course must be laid immediately after the concrete in the bottom course.

The German specifications for roads require a compressive strength of not less than 5260 lb. per square inch and a bending strength of not less than 640 lb. per square inch. Cores taken from the road two months after it is laid must have a minimum strength of 4550 lb. per square inch.

For slabs 8 in. thick laid in lengths of 100 ft., with two dummy joints in this length, 0·1 per cent. of reinforcement is sufficient. In slabs about 300 ft. long, 0·3 per cent. of reinforcement in the middle third is considered to be necessary. Slabs more than 1200 ft. long have been laid with various amounts of reinforcement. The German specification requires joints at intervals of 33 ft. to 50 ft., although greater intervals are permitted in roads laid on firm bases and in districts where the climate is equable. The author's recommendations are that in roads to carry heavy traffic the amount of reinforcement should be from 0·16 to 0·2 per cent. and the slabs from 90 to 125 ft. long. In the case of slabs 300 ft. or more long on sand, about three times as much steel should be used in the middle third. There should be twice as much reinforcement near the sides of the slab as in the middle, with 1½ in. to 2 in. of cover, so as to prevent tensile stresses in the concrete.

The German motor roads are generally 8½ in. thick, without thickened edges, and they have not been damaged by axle loads of 8½ tons. Axle-loads of 10 tons are now permitted; the author doubts the wisdom of this increase because of the ill-effects of the repeated overloading of reinforced concrete slabs.

# Structure at a Gas Works.

New works at Phoenix Wharf, London, S.E., for the South Eastern Gas Board include a structure (*Fig.* 1) for the storage and bagging of sulphate of ammonia. The building is supported on in-situ piles driven through alluvial deposits to gravel about 20 ft. below the surface. There are 213 cast-in-situ piles under the floor of the store carrying a maximum load of 60 tons each, and 176 other piles carrying a maximum load of 45 tons each. The piles under the abutments have a rake of 10 deg.

The store is 168 ft. long by 96 ft. wide and the bagging house 60 ft. long by 108 ft. wide. The floor of the store comprises transverse tie-beams and slabs with a granolithic finish; the floor of the bagging department comprises slabs between beams spanning from the pile-caps.

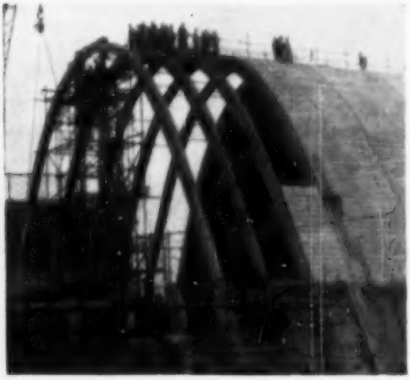


**Fig. 1.—View during Construction.**

The store has sloping reinforced concrete retaining walls 17 ft. 6 in. high cast in-situ with buttresses at 8 ft. centres; opposing pairs of buttresses support prestressed precast three-pin arches with a span of 86 ft. 6 in. and a rise of 48 ft. 6 in. The arch ribs (*Fig.* 4) are 1 ft. wide and vary in depth from 1 ft. 9 in. at the ends to 2 ft. 9 in. at the point of maximum bending moment. Each rib is 66 ft. long and weighs about 13 tons. They were made on a curved concrete base with side shutters that were used several times. The central parts of the arches, 52 ft. 3 in. long, were cast on this bed between two factory-made end-blocks (*Fig.* 3). This method avoided joints in the ribs and allowed the cast iron rocker hinges at the abutments (*Fig.* 5) and the knife-edge hinges at the apex to be incorporated in the end-blocks at the factory.

**Fig. 2.—Erecting an Arch Rib.**

The end-blocks were placed on the casting bed and 1⅛-in. Lee-McCall prestressing bars passed through them; the bars between the end-blocks were in flexible metal tubes. The part of the arch between the end-blocks was then cast. The concrete was required to have a strength

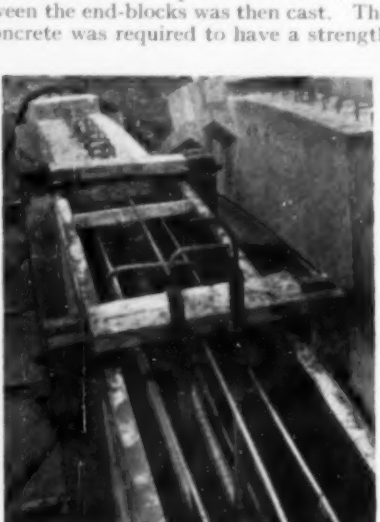


**Fig. 3.—Middle Part of Arch Ready for Concreting.**

of 4500 lb. per square inch at ten days; the proportions were 1 : 1·4 : 2·8 by weight and the water-cement ratio 0·38. Sulphate-resisting cement and aggregate of ¾ in. maximum size were used.

The arches were cast at a rate of five per week, stressing being carried out six days after casting. The bars in each rib were in two lengths of 62 ft. and 5 ft. 0⅞ in. connected with couplers. The arches were erected (*Fig.* 2) with a derrick crane with a jib 117 ft. long.

The roof comprises precast concrete slabs prestressed by a pretensioning process; they are 7 ft. 6 in. long, 1 ft. wide, and 3 in. thick. Steel loops projecting from the ends of the slabs fit over loops projecting from the ribs; mild steel bars

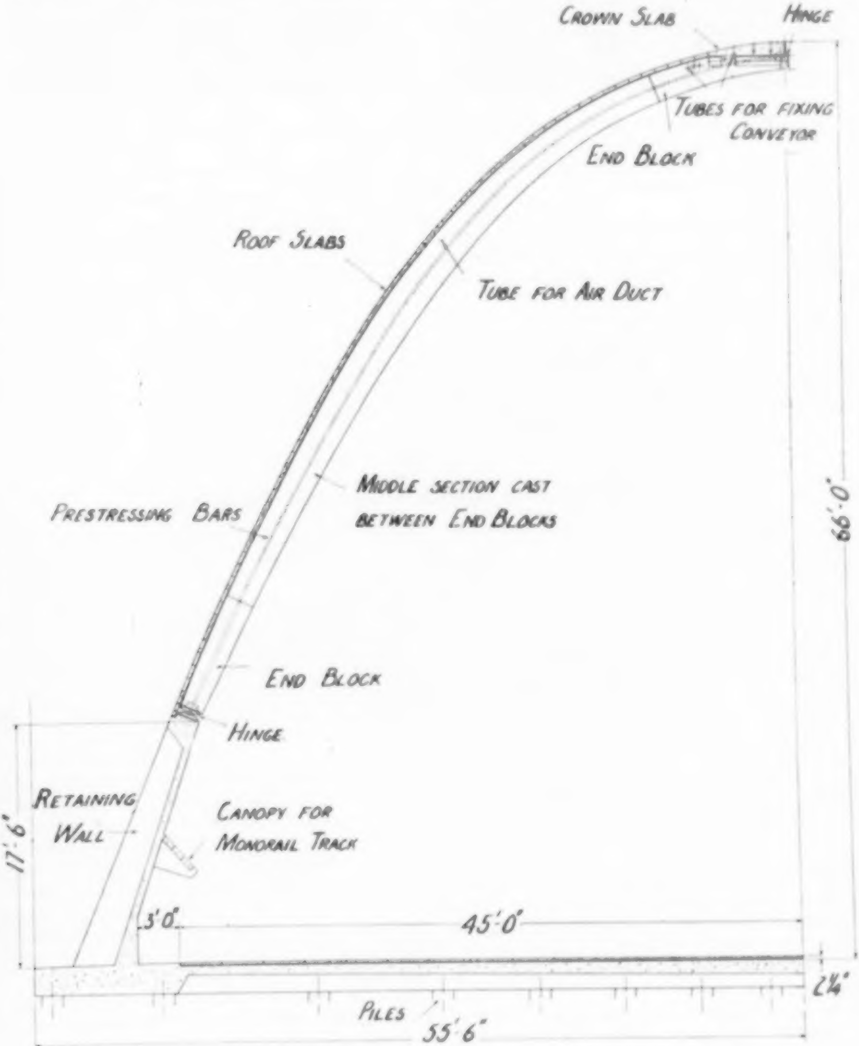



**Fig. 4.—Details of Arch Rib.**

**Fig. 5.—A Rocker Hinge.**

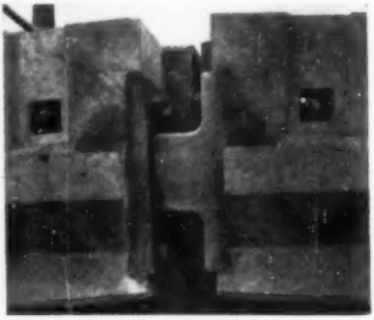


**Fig. 6.—Hinge at Apex.**

were passed through the loops and the space between the ends of the slabs was concreted.

Two precast slabs, resting on corbels, span between the apexes of the arch ribs, and are tied to each other and to the ribs by seven cables and prestressed on the Gifford-Udall-CCL system. The cables are lapped, and span from outside to outside of adjacent ribs. These slabs kept the arches in position while the rest of the roof slabs were erected. The roof of the store has an exterior finish of two layers of bitumen on which crushed white sea-shells were sprinkled.

The structure was designed by the Board in conjunction with Twisteel Reinforcement, Ltd., who prepared most of the detailed drawings. The contractors are the Demolition & Construction Co., Ltd. The precast end-blocks were made by the Liverpool Artificial Stone Co., Ltd., and the roof slabs by Anglian Building Products, Ltd.

## Experiments with Expanding Cement.

THE Swedish Cement and Concrete Research Institute has issued a bulletin (printed in the Swedish language) entitled "Experimentell jäm fördse mellan Lossiercement och standardcement i cementbruk," by Gunnar Lindh, in which are described experiments on the use of expanding cement. The experiments were made to assess the value of the expanding cement invented in France by M. Henry Lossier in the construction of underground tanks for the storage of petrol. It was expected that concrete containing expanding cement would be subjected to an initial compression, with the result that leakage would be prevented and that no metal lining would be necessary. Most of the tests were carried out on specimens made of cement paste and cement mortar. The results of the tests may be summarised as follows.

It is advisable to use a low water-cement ratio if great expansion is desired. For example, the expansion was 2 per cent. with a water-cement ratio of 0·25 and 0·9 per cent. with a water-cement ratio of 0·49. The expansion of cement mortar is about 20 per cent. of the corresponding value for cement paste, and the expansion of concrete mixtures seems to be considerably smaller. However, freedom from shrinkage can always be guaranteed. Curing in water must take place during a relatively long period (not less than ten days) if the concrete is not to be damaged when exposed to frost. The modulus of rupture of expanded cement was lower than, or possibly in a few exceptional cases comparable with, that of ordinary Portland cement. The water-absorption tests indicated that mortar made with expanding cement absorbs more water than mortar made with ordinary Portland cement. If expanding cement is used for the construction of underground tanks for storing petrol, then the water-cement ratio should be low and the water-curing period should be from two to four weeks.

# Warehouse and Workshop at Sunbury-on-Thames

A REINFORCED concrete framed factory and warehouse (*Fig.* 1) for Autolex, Ltd., at Sunbury-on-Thames, is to have a ground floor area of 50,000 square feet of which 45,000 square feet have already been built. The total width of the buildings is 200 ft., comprising outer bays each 60 ft. wide for manufacturing processes and a central bay 80 ft. wide for use as a warehouse. The outer bays are of beam and column construction, the columns being spaced on a grid 20 ft. square. The flat roofs of these bays are 17 ft. above ground-floor level in the central portion and 4 ft. lower at the sides; the difference in the levels is formed by clerestory windows. Secondary beams at 6 ft. 8 in. centres support the roof slabs, the shafts, and the lifting gear; the main beams span between the columns.

The warehouse comprises three-pinned arch ribs at 20 ft. centres (*Figs.* 2 and 3) with longitudinal braces between them; the braces at eaves level are box-shape and are used as a gutter. The roof is covered with corrugated asbestos-cement sheets and plastic glazing supported on steel-channel purlins at 4 ft. 6 in. centres. The hinges at the crown and foundations of the arched ribs are formed by three ⅞-in. diameter rods and filled with a compressible bitumen-fibre board. Joints ½ in. wide between the vertical portions of the ribs and the columns of the side bays allow for expansion of the roof and are similarly filled.


Fig. 1.—Cross Section

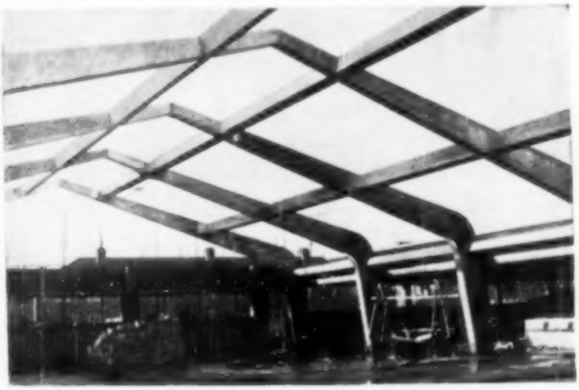


Fig. 2.—During Construction.

After construction of the warehouse had started it was decided to provide space for a canteen within the warehouse which would obstruct the floor as little as possible. This was achieved by making use of part of the space under the roof. On two rows of columns 9 ft. apart a floor with cantilevers at each end was constructed (*Figs.* 3 and 4). This floor is 11 ft. 6 in. above the ground floor and the

cantilevered beams are at 10 ft. centres. Columns at each end of each cantilever support the walls of the canteen. All the concrete was a 1 : 2 : 4 mixture cast in situ.

The architect is Mr. D. A. Grant, B.Arch., A.R.I.B.A. The reinforced concrete was designed and its erection is being supervised by Messrs. Johnson's Reinforced Concrete Engineering Co., Ltd., and the contractors are Moorcroft Construction Co., Ltd.

**Fig. 3.—Canteen to the Right.**

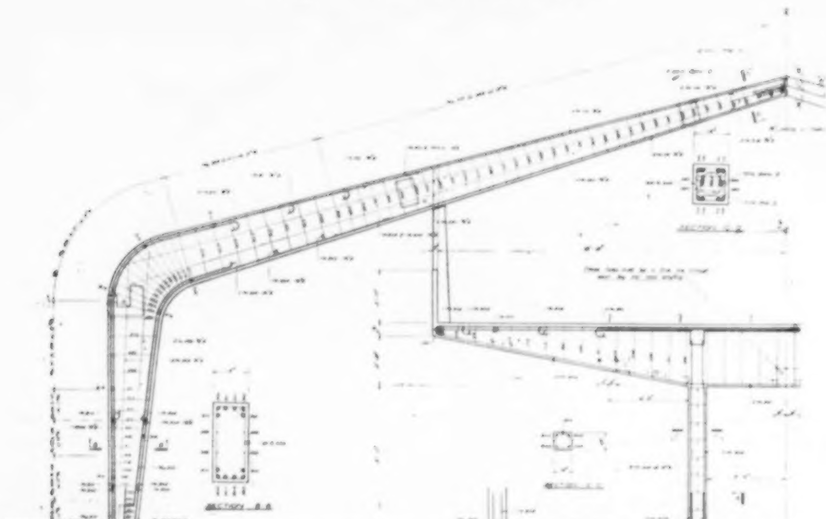


**Fig. 4.—Part Cross Section through Warehouse and Canteen.**

# Renovation of a Deteriorated Concrete Structure.

By W. E. I. ARMSTRONG, T.D., M.Eng., A.M.I.C.E., A.M.I.Mech.E.

THE chemical factory of Commercial Solvents (Great Britain), Ltd. (one of the Distillers Co., Ltd., group of companies), on the south bank of the river Mersey at Bromborough, Cheshire, is a reinforced concrete building 578 ft. long, 130 ft. wide, and 80 ft. high. It was built about thirty years ago, and taken over by Commercial Solvents (Great Britain), Ltd., in 1935. Rusting, and consequent expansion, of the steel reinforcement had occurred on most parts of the exterior and on the soffits of many interior beams and reinforcement in all directions. The rusting of the bottom reinforcement in the interior beams and the roof slabs was due to the corrosive atmosphere entering cracks. The top reinforcement in the beams had not deteriorated except where it was insufficiently covered. The penetration of rain helped the rusting of the reinforcement in the roof slabs. This article describes the renovation work, which was carried out at a cost of over £30,000, to the exterior and some parts of the interior.

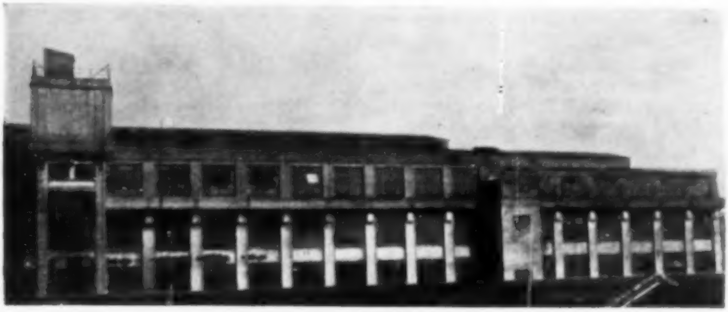


**Fig. 1.—Part of Elevation before Renovation.**

roof slabs, causing the cover in these places to be spalled off (*Figs.* 1 and 2). Further deterioration adjacent to the exposed reinforcement, as well as additional new deterioration, were rapidly taking place. If this deterioration had been allowed to continue the useful life of the building would have been much reduced, and the pieces of concrete which were spalling off would have become a serious danger as their size increased. Renovation was therefore essential.

The rusting of the reinforcement on the outside of the building was caused by two main factors. First, the reinforcement, especially the flat-bar type of stirrup used, had been placed with insufficient cover—often only about ¼ in.; secondly, the atmosphere of the factory was conducive to corrosion. When the steel was exposed the atmosphere and the rain caused rust to spread rapidly along the

### Exterior Treatment.

The major part of the work was the renovation of 85,313 sq. ft. of the external concrete surface by the Aerocem process, by which it was possible to copy the decoration exactly while effectively protecting the steel without adding more weight than the existing structure and the foundations could safely carry. The work comprised cutting out defective concrete, removing rust-scale from exposed reinforcement, cleaning and hacking the remaining concrete and spraying on to the whole a "tac" coat to provide adhesion between the new and old concrete. The surface was then built up in layers to the original profile with cellular concrete before spraying, in layers, ½-in. additional cover. The defective concrete was cut out with chipping hammers, assisted where necessary and permissible by light

pneumatic tools. In some parts of the building the fire-prevention regulations necessitated the use of non-sparking chipping hammers, which were more costly than the normal type and had a shorter life. Cleaning was effected by jets of water under high pressure and wire brushing.

The "tac" coat consisted of a 1 : 2 Portland cement : sand mixture with an emulsion of polyvinyl acetate added in the proportion of 1¼ gall. to 60 lb. of cement, a foaming agent, and water. The cellular concrete consisted of Portland cement and sand in the proportions of 1 : 3 with water and a foaming agent. In both mixtures the sand complied with B.S. No. 1199.

The mixing and spraying apparatus consisted of a high-speed mixer, pressure-pots into which the mixture was transferred from the mixer for transmission under pressure through pipes to the spray-guns, the guns, and a compressor. The compressor provided air for forcing the material from the pot into the pipes and so to the guns and also for spreading out the material coming from the guns into a spray before application to the structure. A close-textured foam was produced in the mixer before the addition of the other materials. The foam consisted of a large number of minute unconnected bubbles which, when evenly dispersed throughout the mixture, improved the watertightness of the concrete and made it more workable. The pressure pots seemed to produce the best results when they were as high as possible, and an electric winch was used to elevate the material to this level; this winch had a specially-enclosed motor to comply with the fire-prevention regulations.

**Fig. 2.—Typical Deterioration**

One-sixth of the building was scaffolded to the full height, and this scaffolding was moved around the building as the work proceeded. Cradles were used for part of the work, but were not very efficient due to the number of moves up, down, and sideways, which were necessary.

Where acid was penetrating through the concrete from the inside (the dark marks in *Fig.* 1) the penetration was stopped by laying a new floor and an interior lining adjacent to the exterior

**Fig. 3.—Part Shown in Fig. 1 Complete on Left. Work in Progress on Right.**

face in concrete made with high-alumina cement, combined with acid-resisting fully-glazed half-round earthenware channels to lead liquid away from the wall. *Figs.* 3 and 4 show the parts shown in *Figs.* 1 and 2 after repair; the retention of the decoration will be noted.

Whilst the scaffold was in position, the opportunity was taken to replace some of the metal window frames which had deteriorated as a result of rust and wind pressure. On the right of *Fig.* 3 are seen the new precast reinforced concrete glazing bars. A new horizontal beam has been inserted to reduce the length of the new bars, and the top openings, which served no useful purpose, have been filled with 4½-in. brickwork containing reinforcement in alternate courses. This brickwork was sprayed on the outside with a ¼-in. thickness of cover. Some metal frames were replaced with precast reinforced concrete window frames with rustproof metal window openings, but these reduced the amount of light that gained access.

Other exterior work included the fitting of about 1200 ft. of 14-in. by 4-in. coping, asphalting to a thickness of ¾ in. (in two layers) of five sections of the roof which were leaking, new channelling and flashings, improvements to the rain-water drainage system, regrading the ash carriageway, and the reconstruction of part of the roadway. The new road is an 8-in. reinforced concrete slab laid on waterproof paper over ash. Since the road is adjacent to the building on one side and to stanchion bases on the other side, the longitudinal joint and the joint alongside the building are expansion joints. In the transverse joints, ¾-in. diameter dowels 2 ft. long and fitted with light metal dowel caps were used at 12-in. centres. The crossfall was directed away from the building. Precast concrete kerbs were laid flat on the slab, with vertical dowels and concrete as rear support.

## Interior Work.

The seriously deteriorated interior beams were treated in the same way as the outside except that No. 8-gauge electrically-welded hard drawn steel fabric with 4-in. square mesh was plugged to the sides and soffits of the beams, and the cover was increased to 1-in. The seriously deteriorated soffits of the roof slabs were treated in the same way as the walls

**Fig. 4.—Part of Renovated Work.**

except that the main mixture was applied as a thin protective layer only, the roofs being waterproofed with asphalt.

As the building is divided internally into sections by walls extending from below the basement floor to the roof, one section was repaired at a time. On some parts of the soffits of the roof slabs, scaling and bush-hammering had to be used to make the surface suitable for spraying.

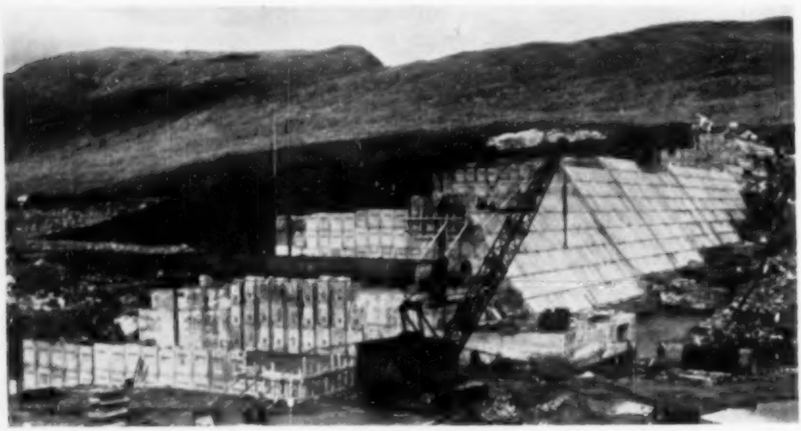
Acknowledgement for permission to publish this article is made to Commercial Solvents (Great Britain), Ltd., to the Chief Civil Engineer of the Engineering Division, Southern Office, of the Distillers Co., Ltd. (who supervised the work), and to Messrs. Mears Bros. (Contractors), Ltd., the main contractors, who also carried out all the spraying.

# Precast Facing Blocks for Dams.

The Moriston project is one of the largest schemes now in progress for the North of Scotland Hydro-Electric Board. The scheme includes two large plain concrete gravity dams, three tunnels, and two underground generating stations with a total capacity of 52,000 kW. An interesting feature is the use of Trief cement; this is a process of making Portland blast-furnace cement on the site, and was fully described in this journal for August, 1954. This cement is probably more resistant than ordinary Portland cement to acid water, and is being used in the tunnel linings as well as in the dams, for which purpose the plant was initially erected.

Another interesting feature is the use of large self-supporting precast blocks in place of shuttering, as shown below. The advantages of this method of construction are that the face of the dam can be formed in high-grade concrete cast on a vibrating table under factory conditions. Also, the blocks may be placed long before the main concrete is placed behind, with a consequent greater flexibility of the concreting programme. On a large project such as this the number of carpenters employed can be reduced considerably by the use of precast blocks for the main facing, and this was an important consideration in deciding on this method of construction.

The contractors for the upper works, comprising two dams, two tunnels, and a generating station, are the Mitchell Construction Co.; for the lower works the contractors are Messrs. Duncan Logan (Contractors), Ltd. The engineers for the whole project are Sir William Halcrow & Partners, MM.I.C.E.



# Correspondence.

## PRESTRESSED SLABS FOR RAILWAY BRIDGES.

MR. W. T. WILKS, of Chorley Wood, writes as follows.

The article in your journal for September, 1954, on prestressed concrete slabs for railway bridges as used on the Western Region of British Railways leads one to the conclusion that in this instance, although prestressed slabs are practical, their economical application has been overlooked, or alternatively has been of secondary consideration. Whilst it is praiseworthy to experiment with new methods and materials in an attempt to reduce maintenance and capital costs, thereby contributing to repairing a serious defect in railway economy, experiments should be directed into channels where there is a wide application for their use if they prove successful. Although prestressed concrete slabs can be used to a degree as decking for bridges, normal reinforced concrete slabs have to be used on skew spans. In this country a high percentage of bridges must have skew spans.

The cost of providing bridges with prestressed concrete slabs must be out of all proportion to the advantages to be derived therefrom. Since special steel jigs have to be provided for making the slabs, the cost is considerably higher than for normal slabs, apart from other minor considerations. Furthermore, the modified design of the main girders to facilitate erection results in heavier girders to resist the torque produced in them and the connections by the eccentric loading. In the connections of both the girders and the slabs secondary stresses are introduced, and although they may be acceptable in steel-framed buildings where only static loads have to be carried, many engineers would be reluctant to adopt them in structures carrying dynamic loading as is the case in bridges.

In comparing the earlier design with a later design, as mentioned in the article, it seems that the designer was conscious of the question of eccentricity when adequate working space was not provided in the earlier design.

It is surprising to find that, in the earlier bridge, the old and undesirable practice was adopted of filling with plain concrete the space between the end of the slab and the main girder. For a good number of years many engineers have endeavoured to keep the main girders as far as possible free; first so that the inspector can see what is happening to this important member of the structure, and secondly to facilitate painting and maintenance. Moreover, the restricted width of the decking in the later design does not give the District Engineer much scope for altering and improving the alignment of the permanent way.

On balance, the disadvantages outweigh the advantages of prestressed concrete slabs for the decks of small-span railway bridges as illustrated in this article. One wonders if this experiment has achieved any positive result, although it cannot be denied that prestressed concrete has a wide application in railway engineering.

MR. P. S. A. BERRIDGE, M.B.E., M.I.C.E., the writer of the article concerned, replies as follows.

The article dealt particularly with the half-through type of plate-girder bridges having a deck consisting of precast prestressed concrete units which could be erected piecemeal quickly and easily with the minimum of interruption of traffic, and which would give long service with very little expenditure on maintenance. The design, produced at a time when steel was in short supply, may appear to be novel inasmuch as the combined use of precast concrete and welded steel girders is concerned, but it cannot be classed as an experiment as Mr. Wilks suggests. The prestressed deck makes use of concrete in its most satisfactory form and provides a bridge requiring the minimum construction depth with normal ballasted permanent way. The design is obviously more suitable for square or nearly-square openings, but, by the use of end units consisting of prefabricated steel joists and reinforced concrete, this type of bridge has been adopted for skew spans.

The prestressed concrete units are relatively cheap to produce and, as the steel jigs are used over and over again in the

production of large numbers of units, their cost is spread over many bridges of the same type. The steel connections to the main girders are designed to carry the stresses due to the end-fixing moments, this being a normal requirement in the design of any half-through type of girder bridge; and, contrary to Mr. Wilks's assumption, the girders themselves, apart from the increased number of stiffeners, are not heavier than would be the case with any other form of deck. Secondary and deformation stresses exist in all through-type girder bridges and, although due allowance has not always been made for them in the past, it is right that they should be considered in this design incorporating prestressed concrete of which experience in railway bridges is as yet very limited. Unlike the steel deck consisting of cross girders and stringers, longitudinal floor-interaction has been eliminated since there is no stress-carrying connection between adjacent units in the deck.

I agree with Mr. Wilks that the filling in situ of the space between the ends of the units and the main girders with plain concrete was undesirable. In the later designs this concrete has been omitted for the very reasons that he has stated. The primary objection to the earlier design was the need for using high-strength bolts of large diameter which could not be readily tightened to the required torque; it was not a question of the working space being inadequate.

As regards allowance for aligning the permanent way, this has been carefully considered in the design of these bridges. In the multiple-track spans, provision is made for the aligning of the tracks at their normal centres.

## THE DESIGN OF LIGHTING COLUMNS.

We have received the following from Mr. F. R. S. Yorke, F.R.I.B.A., a member of the Street Furniture Committee of the Council of Industrial Design, with reference to the Editorial Note in this journal for November, 1954.

"Sir,—I suggest that the appalling muddle of our streets is a very real factor in many road accidents. Any simplification of street furniture, which will make our roads safer as well as more pleasant, will, I am sure, have widespread support. This does not rule out decoration, provided it is subordinate to this consideration. But the quantity-produced column has to fit easily into a variety of situations and alongside many architectural styles. The height of a column is generally dictated by the efficient distribution of light at night, and often makes it seem out of scale during the day, which inept decoration could easily accentuate.

"I can assure you that the Street Furniture Committee of the Council of Industrial Design of which I am a member (or, so far as I am aware, the Council in its wider work) is certainly not opposed to decoration where it is appropriate.

"But, surely, the basic shape of such things as lighting columns, which are often seen outlined against the sky, is the first essential in design. And here I submit that the Council, with the active collaboration of the manufacturers concerned, has been able to make a useful contribution."

These designs were also criticised in "Concrete Building and Concrete Products," and the following is a letter sent to that journal by Mr. George Williams, Secretary of the Street Furniture Committee of the Council.

"In your review of the new designs for lighting columns you remarked on the trend towards simplicity and economy and raised the very pertinent question as to whether the Council of Industrial Design is against decoration. The answer is that the Council is neither 'for' nor 'against' decoration. Each case must be treated on its merits. There could obviously be occasions as in some central urban site where civic pride might well call for some ornamental expression, but the problem of mass production for general use is a very different question.

"In view of the already complicated and distracting confusion of street furniture in most of our thoroughfares simplification becomes a practical necessity. For such cases the Council's Street Furniture Committee favours elegance of proportion and line rather than additional ornament; the multiplication of ornamental columns would only add to the confusion, whereas the repetition of slim columns can be decorative and make a positive contribution to our highways."

[This subject is dealt with in our Editorial Note on page 1 and also on page 20.—Ed.]

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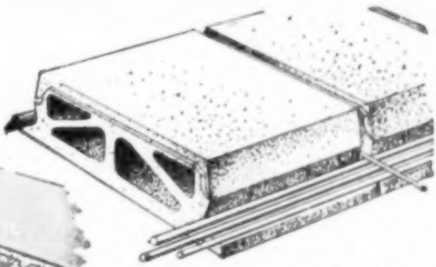

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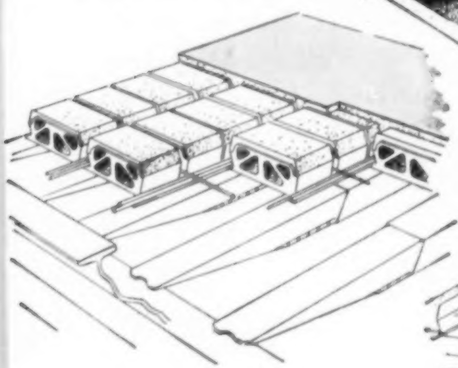

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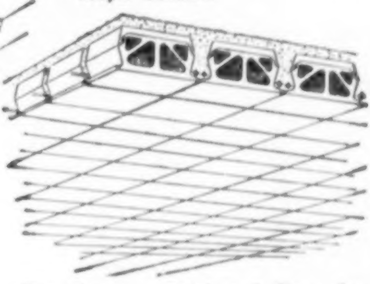

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# Developments in Proportioning, Placing, and Finishing Concrete.

AN investigation into the standard specifications for concrete mixtures was ordered by the Director-General of the Ministry of Works with a view to improving the quality of concrete, economising in cement, and reducing cost by new methods. The investigations started about four years ago and some of the results are now reviewed by the Ministry. The following is an abstract of a report issued by the Ministry.

### Method of Specifying Mixtures.

It was decided that standard specifications were unsatisfactory; these usually called for a nominal mixture, a minimum cube strength, and frequently a definite slump, irrespective of the characteristics of the aggregates used. All the requirements of a specification could not always be fulfilled. It was therefore decided to specify the minimum strength of the concrete and any other requirements such as freedom from surface defects and water-tightness that might be required, leaving the contractor free to proportion the concrete to suit the aggregates available.

Specifications of this type were introduced by the Ministry in 1952, and more than sixty contracts have been completed in which the concrete was specified in this manner. The results are considered so successful that all Ministry contracts in which there is a considerable quantity of concrete now contain specifications of this type. The Ministry advises contractors, if so desired, on the proportions that can be used in any particular case. In general, the results show that this type of specification, using mixtures proportioned for workability and strength to suit the aggregates, and compaction by vibration, can lead to a reduction in costs and to improvement in the uniformity of the quality of concrete.

The reduction in the proportions of cement used has varied from about 10 per cent. in hand-placed concrete with a compressive strength of 3000 lb. per square inch at 28 days, to 40 per cent. in high-quality concrete for prestressed work. Reductions in cement content of 15 to 20 per cent. are common when vibration is used to produce concrete with a minimum strength of 3000 lb. per square inch at 28 days. Examples are:

(1) An office building with basements 500 ft. long. The concrete in the basements was in the proportions of 1 : 2·23 : 6·77 by weight with an aggregate of 1½ in. maximum size to ¾ in. The average cube strength was 4006 lb. per square inch at 28 days. The required minimum strength was 3000 lb. per square inch at 28 days and the normal mixture would have been 1 : 2 : 4 by volume. The saving was 0·93 cwt. of cement per cubic yard, or 20 per cent.

(2) For several large reinforced concrete structures the required strength at 28 days was 4000 lb. per square inch. The nominal mixture would have been 1 : 1½ : 3 by volume. The mixtures used varied from 1 : 1·87 : 4·38 by weight, resulting in a saving of cement of 13 per cent., to 1 : 2 : 6 by weight showing a saving of 30 per cent., depending on the quality of the aggregates available.

(3) In a prestressed structure the required strength was 5500 lb. per square inch at 28 days. The mixture used was 1 : 1·74 : 4·87 by weight, having an average 28-days' cube strength of 6660 lb. with a saving in cement of 40 per cent. compared with a nominal 1 : 1 : 2 mixture by volume.

Savings of 15 to 20 per cent. of cement are normal. As a rule the characteristics of the coarse and fine aggregates available control the mixture that can be used in any particular case.

Apart from the saving of cement the advantages obtained from these mixtures are (a) The saving of a few shillings per cubic yard in the cost of materials, (b) the production of concretes that are denser, less liable to damage by frost, and less liable to cracking and crazing due to shrinkage, and (c) it is not uncommon for such a mixture to have better workability than a standard mixture even though it has a lower water-cement ratio.

### Consolidation by Vibration.

The full advantages of such mixtures are obtained only if vibration is used, but there are similar though lesser advantages

in such mixtures placed by hand. If concrete is to be hand-compacted it is seldom necessary to use a mixture richer than 1 : 7½ by weight in order to obtain a minimum cube strength of over 3000 lb. per square inch at 28 days. Such mixtures will show a saving of cement of up to ½ cwt. per cubic yard.

Provided that concrete of reasonable quality is needed (that is a concrete with a water-cement ratio below 0·6) it will usually be cheaper to use internal vibration. A good type of internal vibrator can consolidate thoroughly more than 5 cu. yd. per hour, and it has been found economical to use 1½-in. aggregate for work in which this type of aggregate would not be permitted by the Code of Practice (for example, where the cover is less than 1¾ in.).

In the course of this work some processes have been investigated which promise reductions in cost, namely, a power float for surface finishing; a vibrating roller for consolidating thin concrete slabs on ground; and striking shutters while the concrete is green.

POWER FLOAT.—This machine (*Fig.* 1) was originally used for laying granolithic paving. It necessitated the use of a lower water-cement ratio and a smaller proportion of sand than are used with hand-placed granolithic, so that there is less laitance to cause dusting. The machine also produces considerable consolidation, resulting in a harder and more durable surface. The machine will do as much work as eight plasterers in a given time. A further development is to lay the granolithic immediately after the

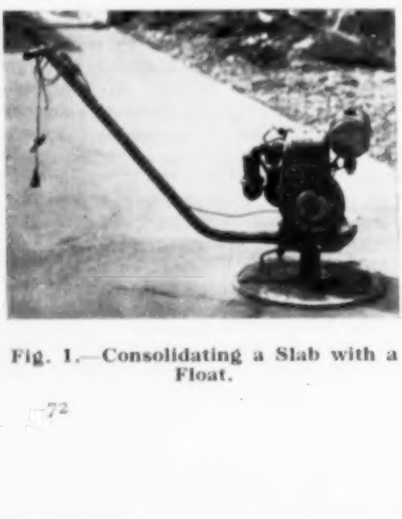

**Fig. 1.—Consolidating a Slab with a Float.**

**Fig. 2.—Vibrating Roller.**

base has been compacted by vibration, when the pressure of the machine knits the topping to the base and the thickness of the granolithic need be only ½ in. or ¾ in. As the two layers are fully monolithic, the granolithic can be considered as part of the structural member and the total thickness reduced by this amount. There is no possibility of the granolithic surface lifting, and tests show that there is no tendency for the topping to separate from the base under load. The machine can also be used for producing a finished surface on an ordinary concrete slab, and it has been used to finish 1 : 9 concrete with aggregate of 1½ in. maximum size.

VIBRATING ROLLER.—The vibrating roller (*Fig.* 2) has been found satisfactory in consolidating ground slabs up to 8 in. thick, and with a wide range of mixtures. Full consolidation throughout the thickness of the slab has been obtained with lean dry concrete in the proportions of 1 : 16; the crushing strength of 6-in. cubes cut from this slab was over 2000 lb. per square inch at 28 days.

With mixtures of about 1 : 8 the strength of cores has regularly been over 5000 lb. per square inch at 28 days, and where the strength of the concrete is the determining factor it has been possible to reduce the thickness of the slabs from 6 in. to 5 in.; where a power float has been used in conjunction with the roller, slabs which would normally be 6 in. thick including a granolithic topping are 5 in. thick including ½ in. granolithic.

Using a carefully proportioned and vibrated concrete it has been possible to strip shutters about two hours after placing. It is possible to strip them much earlier than this, but this is not considered desirable.

# A Large Prestressed Roof in Rome.

IN a recent number of the Italian journal "L'Industria Italiana Del Cemento" a description is given of a large dining hall constructed in Rome with a covered area of about 1800 sq. yd. without intermediate supports (*Fig.* 1). The roof consists of prestressed two-pinned portal frames with a theoretical span of 106 ft. 7¾ in., a total height of 23 ft. 1½ in., and a spacing of 15 ft. 10 in. (*Fig.* 2). The frames support a reinforced concrete hollow-tile slab and are designed for a superimposed load of 30 lb. per square foot.

The legs of the frames are of variable cross section and are pin-jointed at the base immediately above the pile caps. The foundation comprises groups of bored in-situ concrete piles. The outer piles of each group are inclined to resist the horizontal thrust from the frame. The beam of each frame is of hollow rectangular section with two sides each 3½ in. thick joined together at the top and bottom by in-situ slabs 4¾ in. and 3½ in. thick respectively. The webs were precast in sections and hoisted on to temporary supports by a derrick crane. Holes through the webs for the passage of the prestressing cables were formed by bars which were withdrawn a few hours after casting.

The concrete mixture was 1 cwt. of cement to 4·7 cu. ft. of combined aggregate. The prestressing cables each consisted of sixteen 5-mm. diameter wires. Six cables are used in each beam and four in each leg of the frames.

The sequence of operations was: (a) To tension the cables gradually to avoid loss of the prestressing force due to elastic shortening of the beam and to reduce the losses due to internal friction on the cables. (b) Three months after the first tensioning, proceed with the differential tensioning to eliminate the losses due to creep. The cable holes were then grouted. As the elastic shortening of the beams due to the prestressing force reduced horizontal thrust at the joints, with a consequent decrease in the end-fixity of the beams and because, in addition, further shortening of the beams was caused by the closing of the construction joints in the precast webs, temporary joints were formed between the beams and

Fig. 1.—Interior View.

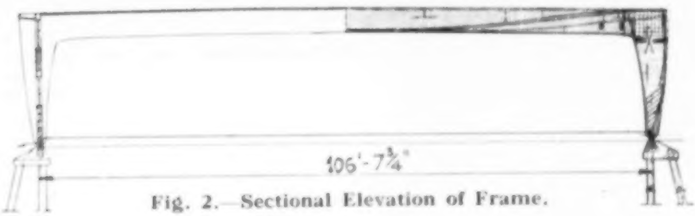


Fig. 2.—Sectional Elevation of Frame.

the heads of the legs in such a way that the prestressing of the beams would not cause loss of thrust. Soon after prestressing the beams these temporary joints were grouted and the legs of each frame were prestressed simultaneously. The temporary supports were then removed. The moments on the frames are largely influenced by the width of the compression flange of the beam formed by the roof slab between the frames, and longitudinal joints parallel to the beams were formed in the roof slabs to establish a definite width of flange in order that the relative stiffnesses of the members of the frame could be determined more accurately and the moments and forces calculated. Results of tests carried out during construction, after the withdrawal of the temporary supports, and under load were very close to those obtained by calculation.

The work was designed and supervised by Sr. Riccardo Morandi and the contractors were S.A. Fratelli Giovanetti, Rome.

# Prestressed Concrete with Pre-tensioned Wires.

### AN AMERICAN SPECIFICATION.

THE following is an abstract of a specification produced by the Prestressed Concrete Institute, of Lakeland, Florida, U.S.A. This is a national organisation in the U.S.A., and we are informed that the specification is being followed by members of the Institute in the production of large quantities of prestressed floor slabs, piles, beams, and other products.

#### Materials.

*Stranded Wire.*—All stranded wire shall be of the 7-wire type having one central wire and six outside wires. The central wire shall be sufficiently larger than the outside wires to ensure that each of the outside wires will bear on the central wire, thus gripping it. All strands shall be stress-relieved as a unit after the wires have been formed into a cable. The properties of stranded wire shall conform to the following:

| Diameter of Cable (in.) | Approximate area (sq. in.) | Minimum ultimate strength (lb.) |
|---|---|---|
| 3/16 | 0·0214 | 5,500 |
| 1/4 | 0·0356 | 9,000 |
| 5/16 | 0·0578 | 14,500 |
| 3/8 | 0·0799 | 20,000 |
| 7/16 | 0·1089 | 27,000 |
| 1/2 | 0·1438 | 36,000 |

Minimum 0·2 per cent. proof stress equals 0·85 per cent. of ultimate stress. Minimum elongation in 10 in. equals 4 per cent.

*Wire.*—All wires shall be of the stress-relieved type and not larger than 1/4 in. diameter. Their properties shall conform to the following: Minimum ultimate strength, 250,000 lb. per square inch; minimum 0·2 per cent. proof stress, 0·8 per cent. of ultimate stress; minimum elongation in 10 in., 4 per cent.

*Concrete.*—The concrete shall have the strength called for on the plans and shall be manufactured, transported, and deposited in accordance with the latest recommended practices of the American Concrete Institute. Air-entraining cement or admixtures may be used to increase workability. The size of the coarse aggregate shall meet the spacing requirements of the prestressing steel and in no case shall be larger than 1 in.

#### Design Stresses.

*Stranded Wire and Plain Wire.*—Initial stresses shall not exceed 70 per cent. of the minimum ultimate strength for stress-relieved stranded wire or plain wire. Loss of initial prestress due to creep, shrinkage, and plastic deformation shall be assumed to be not less than 16 per cent.

*Concrete.*—Maximum allowable stresses in the concrete at the time of the transfer of the prestressing force shall be as follows: Compression in bridge members, 0·50 $fc'$; compression in building members, 0·55 $fc'$; tension, 0·06 $fc'$ (unless additional is resisted by reinforcing steel). Maximum allowable stresses under final dead and live load conditions shall be as follows: Compression in bridge members, 0·40 $fc'$; compression in building members, 0·44 $fc'$; tension in bottom fibre in bridge members, 0·00'; tension in bottom fibre in building members, 0·05 $fc'$; tension in top fibre, 0·04 $fc'$ (unless the additional is resisted by reinforcing steel, but not more than 0·08 $fc'$); diagonal tension, 0·04 $fc'$. When concrete made with lightweight aggregate is used, data on stress losses due to creep, shrinkage, and plastic deformation should be presented, and these losses used instead of those listed in the specification.

[$fc'$ is the compressive strength at 28 days tested on cylinders.]

DESIGN DETAILS.—The spacing of stranded wire cables and plain wire shall be the largest of (1) The centre to centre distance of wires shall be not less than three times the diameter of the wire; (2) the centre to centre distance of stranded cables shall be not less than four times the diameter of the stranded cables; in either case, the clear space between the cables or wires shall be not less than one and one-half times the maximum size of the coarse aggregate.

The minimum distance from any concrete face to the centre of a wire or cable shall be three times the diameter of the wire or cable or one-half its diameter plus 1 in., whichever is greater.

**Manufacture.**

Cables or wires may be tensioned and anchored all at once or one or more at a time at the discretion of the manufacturer.

When two or more cables or wires are tensioned simultaneously, means approved by the engineer shall be provided to obtain as equal tension in each strand or wire as is practical.

For stress-relieved cables or wire, the pretensioning force shall be determined either by elongation based on the modulus of elasticity of the cable or wire or by the load measured by a calibrated gauge, or by both.

Moulds are preferably of permanent type made of steel or concrete. Wooden moulds to produce a smooth finish may also be used.

At least three standard test cylinders shall be prepared at the time the concrete is deposited for each production line, to determine the strength of the casting at different ages.

Pre-tension in the cables or wires shall be released from the anchorage gradually and simultaneously. Unless otherwise approved by the engineer, the transfer of the prestressing force shall not be done until the concrete has a minimum strength of 4000 lb. per square inch.

The moulds shall be designed so that they will not restrict the longitudinal movement of the casting when the prestressing force is transferred.

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*Situations Wanted, 3d. a word: minimum, 7s. 6d. Situations Vacant, 4d. a word: minimum, 10s. Other miscellaneous advertisements, 4d. a word: 10s. minimum. Displayed advertisements, 30s. per column inch. Box number 1s. extra. The engagement of persons answering these advertisements is subject to the Notification of Vacancies Order, 1952.*

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SITUATION VACANT. Reinforced concrete designer-draughtsman required by ASHMORE, BENSON, PEASE & Co., Stockton-on-Tees. Applicants should be fully experienced in designing and detailing reinforced concrete structures, foundations, and other civil work. Apply stating age, experience, etc., quoting Reference D, to Staff Personnel Officer.

SITUATIONS VACANT. Reinforced concrete detailers required by consulting civil and structural engineers. Five-days' week. Berkhamsted, Herts., area. Write, stating age, experience, and salary required. Box 4098, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

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Applicants must have a sound knowledge of good general building construction, together with considerable experience in the supervision of reinforced concrete structures, and should be well acquainted with the quality control of concrete.

Preference will be accorded to applicants who are Chartered Civil or Structural Engineers.

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(Continued on page cii.)

## MISCELLANEOUS ADVERTISEMENTS.

(*Continued from page* ci.)

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SITUATION VACANT. Designer-detailer (reinforced concrete) required by consulting engineers, Westminster. Salary, based on experience and technical education, would be £600 upwards. Post progressive both for position and remuneration. Details of experience, employment, and age to Box 4100, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

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SITUATION VACANT. Draughtsman (civil engineering) required by consulting engineers in Westminster. H.N.C. studies as basis of technical education. Previous experience of reinforced concrete advantageous. Salary will be according to experience. Box 4110, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Clarke, Nicholls & Marcel, consulting engineers, require for their Bristol office reinforced concrete designer-draughtsmen and detailers. Salary in accordance with experience. Reply to Berkeley Cottage, Berkeley Square, Bristol, 8.

SITUATIONS VACANT. Draughtsmen, designer-draughtsmen, and tracers. A number of vacancies in each grade occur on move to new offices in St. Albans by consulting engineers. Write, giving particulars and salary required, to Oscar Faber & Partners, 1 Worley Road, St. Albans.

SITUATION VACANT. Senior assistant required by consulting engineers as reinforced concrete designer, preferably with experience in sewerage and sewage disposal works. Must be fully conversant with design of water-retaining structures. Salary according to qualifications and experience. Pension scheme. Apply with full particulars, including names of at least two references, to J. D. & D. M. Watson, MM.I.C.E., 18 Queen Anne's Gate, London, S.W.1.

SITUATIONS VACANT. Reinforced concrete designers and detailers required for varied and interesting work in N.W. London. Permanent pensioned employment. Five-days' week. Staff canteen. Apply in writing stating age, experience, and salary expected to Personnel Manager (W.1), John Laing & Son, Ltd., London, N.W.7.

SITUATIONS VACANT. Designer-detailer and detailer experienced in reinforced concrete. Knowledge of steelwork an advantage but not essential. Varied and interesting work with scope for initiative. No Saturdays. Good salaries to right applicants, who should apply stating full particulars to Engineering Department, Farmer & Dark, Romney House, Tufton Street, London, S.W.1.

SITUATIONS VACANT. Consulting engineers in Central London require two or three good reinforced concrete detailer-draughtsmen for coal bunkers and similar interesting structures. Five-days' week and excellent office conditions. Write, stating age, experience, salary, and qualifications, to J. C. Hughes & Partners, 83 Gloucester Place, London, W.1.

SITUATION VACANT. Situation vacant with consulting engineers in Central London for a reinforced concrete structural engineer of experience to assume responsibility of drawing office. Good salary and five-days' week with permanent position to suitable applicant. Write, stating age, experience, and salary required, to Box 4111, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Reinforced concrete detailer required for office in Golders Green. Five-days' week. Superannuation and bonus scheme. Reply Box 4112, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Clarke, Nicholls & Marcel require urgently at their London, Bristol, and Leeds offices designers, draughtsmen-detailers, and junior assistants in reinforced concrete and structural steelwork. Permanent positions, good prospects. Apply, stating experience and salary required, to 21 Westbourne Grove, London, W.2. Bayswater 6816.

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1. ASSISTANT STRUCTURAL ENGINEER, GRADE IV, Salary £675 × £30 to £825 p.a., plus London Weighting. Must be experienced in design and detailing of reinforced concrete building frames and/or structural steelwork. Preference given to those who have passed final examination of Inst. of Civil or Structural Engineers.
2. STRUCTURAL ENGINEERING DRAUGHTSMAN, GRADE II, Salary £560 × £20 to £640 p.a., plus London Weighting. Must be experienced in detailing reinforced concrete building frames. Knowledge of structural design would be an advantage.
3. STRUCTURAL ENGINEERING DRAUGHTSMAN, GRADE I, Salary £500 × £20 to £580 p.a., plus London Weighting. Must be neat draughtsman and have had some experience of reinforced concrete or steelwork detailing.

**Applications giving full details and present salary, accompanied by copies of three recent testimonials, to County Architect, County Hall, Kingston, by 22 January, 1955.**

### ASSISTANT TO TECHNICAL MANAGER

Applicants should preferably be young graduates in civil or structural engineering with a good knowledge of concrete technology. The duties involved will be mainly concerned with technical service work and collaboration on development work at the laboratories. This is a responsible Head Office post (address below) with excellent prospects, and a good salary will be offered.

Upper age limit about 25 years. Applications to:—

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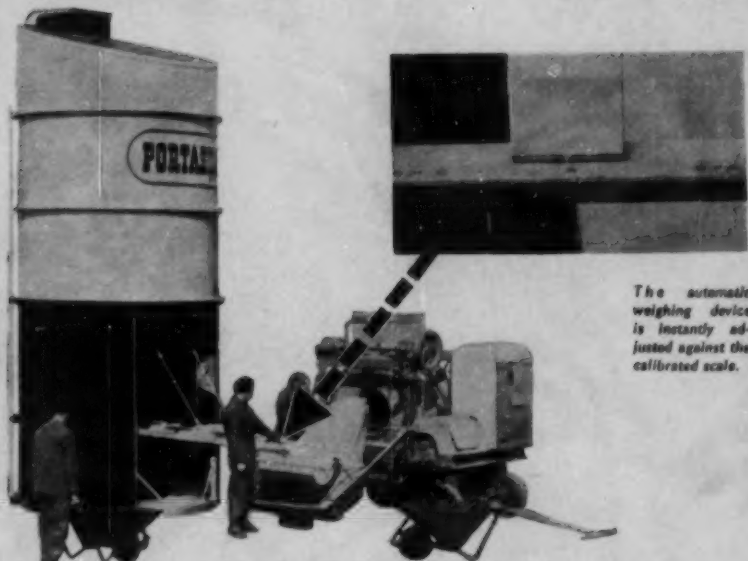

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